JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

VOL. 26 AUGUST, 1934 No. 8

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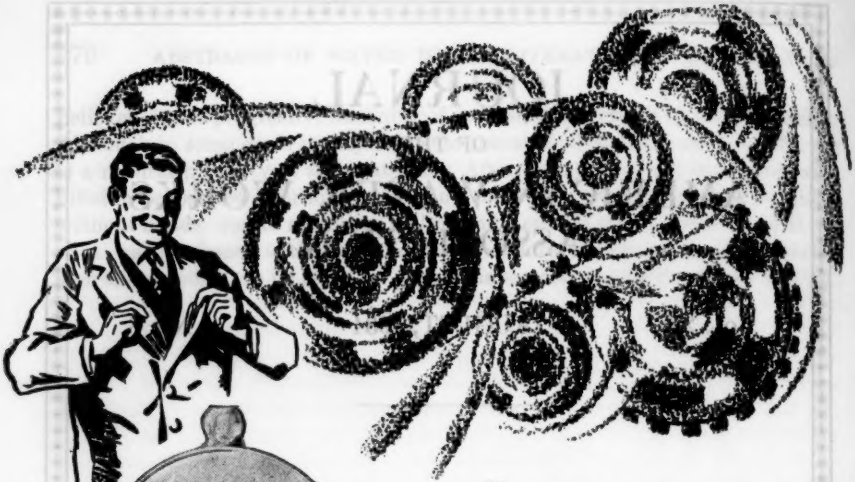

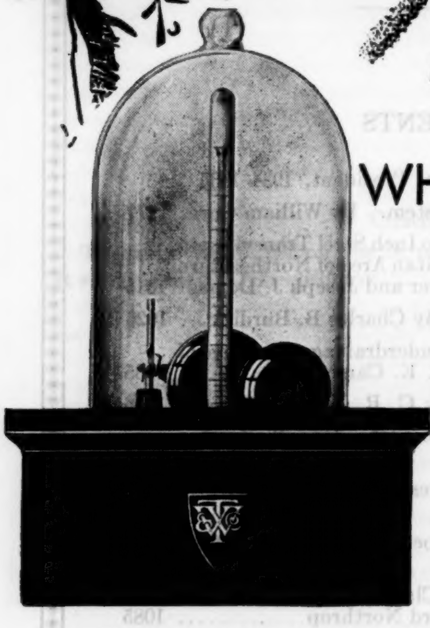

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HARRY E. GORDAN
PRESIDENT, 1934–1935

JOURNAL

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AMERICAN WATER WORKS ASSOCIATION

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Vol. 26 AUGUST, 1934 No. 8

CALGARY'S NEW WATER WORKS SYSTEM

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Early in the year 1933 the City of Calgary put into operation a new water works system. The rapid growth of Alberta's business centre had far outdistanced the development of the water supply and after a heavy flood in 1929, which caused serious difficulties in maintaining the supply of water, it became evident that early steps had to be taken to remedy this state of affairs.

Calgary was so named in 1876, the name being of Gaelic origin, meaning "Clear Running Water." The City is beautifully situated in the foothills of the Rocky Mountains and is often referred to as "The Sunshine City of the Foothills." Located at the junction of the Bow and the Elbow Rivers the City is indeed picturesque, with its distant view of the Rockies lying to the west.

With a population of 500 in 1883 the City's growth has been rapid until at the present time some 85,000 persons live within its boundaries.

HISTORICAL

The original water works system was built in 1889. Just previous to the commencement of the installation of the new system the water supply came from two sources, as follows:

1. A gravity supply of 8,000,000 gallons[1] daily fed at a uniform rate

[1] Note: Wherever the word "gallon" is used in this paper the Imperial gallon is referred to.

from an intake situated on the Elbow River $10\frac{1}{4}$ miles from the City and delivering water through a 30-inch diameter wood stave pipe to two open service reservoirs of 16 and 20 million gallons capacity with the top water level at elevation 3,620 feet.

2. A pumping supply from the Bow River in two parts:

(a) An intake and pumping station, referred to as Pumping Station No. 2, provided with two 7,500,000 gallon electrically driven pumps delivering water into the same mains as supplied by the service reservoirs at a working pressure of 85 pounds per square inch, which is equivalent to the static pressures from the service reservoirs. This pumping station is situated on the right bank of the Bow River where the most important parts of the City are also located.

(b) An improvised pumping station and settling tank on an island upstream from Pumping Station No. 2, with an intake in the Bow River provided with one 8,000,000 gallon electrically driven pump delivering water at a pressure of 85 pounds per square inch to a 30 inch diameter steel pipe and supplying the service mains on the left or north side of the Bow River. These service mains are connected by pipes passing through tunnels underneath the Bow River to the service main on the right bank of the river and to the service reservoirs, as well as Pumping Station No. 2.

The principal difficulties experienced in operating the old system were:

1. Both the Bow and Elbow Rivers are shallow, rapidly flowing streams and during certain winter seasons the running water became filled with floating or frazil ice. This ice choked the intakes and screens which were kept in partial operation at such times only by extraordinary efforts.

2. During periods of high flow the water was very turbid, particularly in the Bow River. This turbidity was reduced in the gravity supply from the Elbow River by using the service reservoirs as settling reservoirs, but the water from each source during high discharges was quite unsatisfactory because of high turbidities.

3. The raw river waters frequently showed contamination of intestinal origin. To provide for the sterilization of the water to meet the requirements of the Public Health authorities it was necessary to add excessive amounts of chlorine, which resulted in disagreeable tastes and well founded complaints.

4. During high discharges of both rivers their banks and beds were

eroded and there was considerable movement of eroded materials and changes in the channels which formed the dry weather beds of the rivers. This was more noticeable in the case of the Bow River, but had been more serious in the case of the Elbow River as the inlet to the gravity intake was on several occasions left high and dry.

5. The gravity main 10¼ miles long from the Elbow River traversed the flood plane of the river for long distances. It was a common occurrence for the gravity main to be undermined by the floods and partially broken, permitting considerable leakage and waste of water and necessitating costly repairs.

6. The gravity main was a wood stave pipe of 30 inches internal diameter, constructed in 1907. Independent of the fractures in it, due to undermining, decay had set in, particularly at the ends of the staves, and the useful life of the main had drawn to a close.

7. A number of the service mains in the city are thin steel pipes which have become corroded through and from time to time are being replaced with cast iron pipes. It is possible that the high consumption of water may in part be due to leakage from these steel pipes.

8. The feeder mains are inadequate and considerable extensions are necessary immediately for present as well as future requirements in order that adequate pressures may be maintained for ordinary and fire services.

9. The almost complete absence of master meters and of divisions or districts of supply makes the control of service conditions difficult.

10. The higher areas were inadequately supplied with storage tanks and although the lower areas had sufficient storage the lining of the 16 million gallon reservoir was in need of repair. The 20 million gallon reservoir also needs lining because wave action upon the soft banks increases the turbidity.

11. All of the pumps were working at times to their full capacity so that the failure of one pump or of the electric power supply would cause a stoppage in the supply of water to many parts of the city over long periods. Furthermore some of the pumps were inefficient and had almost reached the limit of their effective service.

CITY TOPOGRAPHY

The city is intersected by the deep valleys of the Bow River and its two tributaries, the Elbow River and Nose Creek, together with numerous small water courses or coulees. The lowest portion of the

city at the southeast corner has an approximate elevation of 3,300 feet above sea level and the maximum level is approximately 3,875 feet at the northwest corner, or a maximum difference in elevation in the city of approximately 575 feet. The ground surfaces on which the city is built are therefore severed, somewhat irregular and wide spreading, and in consequence the water distribution problem is an unusually difficult one.

TABLE 1

Chemical analyses of Calgary water

(Results in parts per million)

	BOW RIVER	ELBOW RIVER
Residue on evaporation	212	254
Loss on ignition	77	195
Fixed residue	135	159
Silica SiO_2	14.4	9.7
Alumina Al_2O_3	1.5	4.3
Iron oxide Fe_2O_3	0.1	—
Lime CaO	71.4	76.7
Magnesia MgO	25.5	26.7
Sulphate SO_4	47.1	74.8
Chloride Cl	Nil	Trace
Nitrate NO_3	Trace	1.0
Carbonate CO_3	Nil	—
Bicarbonate HCO_3	179	171.9
Bicarbonate alkalinity $CaCO_3$	147	140.8
Carbonate alkalinity $CaCO_3$	Nil	Nil
Hydroxide alkalinity $CaCO_3$	Nil	Nil
Total hardness (soap method) $CaCO_3$	185	
Total hardness (soda reagent) $CaCO_3$	181	200.8
Total hardness (analysis) $CaCO_3$	194	208.8
Noncarbonate hardness $CaCO_3$	33	54.8
Temporary hardness $CaCO_3$	70	—

POPULATION AND AREA

The population of Calgary at the present time is estimated to be approximately 85,000 persons. The area of the city is given as 25,368 acres or 39.64 square miles. Based on this estimated population the density is 3.35 persons per acre. Calgary at the time the new scheme was undertaken had a smaller population per acre than any other city in Canada, with the exception of Edmonton.

DISTRIBUTION SYSTEM

The distribution system as at 1929 consisted of 201 miles of mains, made up of 93 miles of steel pipe varying from 6 to 24 inches in diameter, $1\frac{1}{3}$ miles of wood stave pipe of 6 and 8 inches in diameter and 107 miles of cast iron pipe varying from 4 to 18 inches in diameter. In addition to that the city had a wood stave gravity intake pipe of a total length of $10\frac{1}{4}$ miles and 30 inches in diameter.

NEW WATER WORKS SYSTEM

In July 1929 the writer's firm was called in to make a study of the situation and to report on the whole of the city requirements so as to provide an adequate water supply for the increasing needs of the population. A report covering the situation was submitted under date of October 15, 1929, and the following month the rate-payers authorized the construction of what is now known as the Glenmore Water Works System, involving an expenditure of four million dollars. The report in general indicated the old system was in a very precarious condition and that the city had so far been fortunate in avoiding a major disaster.

A study of the physical character of the Calgary area showed the probable sources of supply to be as follows:

1. Underground sources.
2. The Bow River.
3. The Elbow River.

The underground sources were rejected as a possible source of supply due to the fact that the quantity of water in the river gravels would be entirely inadequate for a growing community like the City of Calgary.

Careful investigations were made of several wells being used for industrial purposes in the Calgary area, the amount drawn being approximately one million gallons per day, and so far as could be judged the water obtained from these wells was satisfactory for industrial purposes.

Extensive consideration was given to the Bow and Elbow Rivers as a source of the permanent water supply for the city. These rivers derive their waters principally from the melting snows of the Rocky Mountains and as has been shown they have similar chemical characteristics. Both are well mineralized and somewhat hard. The flow in both rivers had been gauged in Calgary during the previous 18 years by the Irrigation Branch of the Dominion Water Power and Reclamation Service of the Department of Interior. These gaugings show small

flows during the winter months and large flows during the summer months. The principal flow characteristics were as follows:

	MILLION GALLONS PER DAY		
	Maximum flow	Average flow	Minimum flow
Bow River	29,160	1,970	194
Elbow River	7,668	220	10

As Calgary is situated comparatively near the Rocky Mountains the possibility of an upland supply capable of delivering water to the higher parts of the city, without pumping, demanded the first consideration. Observations showed that both rivers come down from sufficient elevation for this purpose. The level to which the water must be supplied in the city is in the neighbourhood of elevation 3800 feet. In order to be safe at all times it becomes necessary to provide two lines of pipes each of half the full capacity. To drive the water through the pipes an hydraulic gradient of say 10 feet per mile must be provided so that at 10, 20, 30 and 40 miles distant the necessary elevations of the sources must be not less than 3900, 4000, 4100 and 4200 feet respectively. Following up the rivers it was found necessary to go over 50 miles up the Bow and 30 miles up the Elbow in order to secure such elevations. Without discussing such schemes in detail it will be seen that the distances are considerable and that the costs for the pipes or aqueducts to supply an average of 40 million gallons daily, whichever of the two might prove the more economical, would be so high as to more than tax the resources of the City. We were forced, therefore, to study less costly schemes nearer the city in which pumping is required in whole or in part. In the case of the Elbow there were no important industries which affect or may affect the flow of the water in the river. In the case of the Bow the interests of the Calgary Power Company, with its two existing dams near Seebe and the Ghost Project, would affect the flow in that river. Any intake above the Ghost Power development plant would prejudicially affect the interests of the Power Company so that the highest point up the Bow at which water could be taken without interfering with the power interests is near the downstream area of the Ghost Dam at a distance of about 38 miles from Calgary. Arrangements could possibly have been made to take water from the Ghost Reservoir as it would be more economical to take water at the higher head than to pump it up by electric power at Calgary derived from

the same source. Such a scheme was worked out and is referred to later.

The low water stage of both rivers is during the winter months and as most of the water comes from the mountains it is unlikely to be greatly different in quantity during this period at any point along those portions of the rivers studied for water works purposes so that the Calgary gaugings were used throughout our investigations. The regulation of the Bow River for power purposes would tend to increase the river flows in winter but during the past 18 years there has been a flow of not less than about 5 times the required demand of 40,000,000 gallons per day. The flow of the Elbow has, however, frequently fallen below the required amount and storage of water upon it would become necessary for this purpose. On the basis of the winter of the years 1922–23, which was the lowest on record, the net available storage required is 145 million cubic feet, which is 906 million gallons, or nearly 23 days' supply. This volume must be in excess of that occupied by ice and deposited materials and sufficiently large to prevent interference at the intake by frazil ice. It was estimated that at least 60 days' supply should be provided.

PHYSICAL CONDITIONS

The most serious problem that had to be met in the taking of water from either of these rivers is the formation of frazil ice in the running waters. This problem can be solved by the provision of adequate storage of still water which quickly freezes over and prevents the formation of frazil ice. The storage, however, must be considerable but within limits the smaller the river the less the necessary storage, so that from this viewpoint an intake and reservoir on the Elbow was preferable to that on the Bow. During flood periods the rivers become very turbid and settling reservoirs are desirable to reduce the amount of turbidity at the intakes. This storage must be considerable both in order to settle out the solids and to provide volume for them to settle in at small cost. The same storage required for the prevention of frazil ice will also serve for settling. From this viewpoint the Elbow River was also preferable to the Bow owing to the average flow of the Bow being nine times that of the Elbow.

If storage were to be provided on the river itself or a dam of any sort placed thereon it had to be of such form and strength as would permit of the passage of the maximum flood waters ever likely to be encountered. In this respect the Elbow had much the advantage.

Careful examination of both rivers showed better available sites for storage reservoirs upon the Elbow than the Bow. Upon the banks of the Bow are numerous centres of population while the main transcontinental line of the Canadian Pacific Railway runs alongside of it not far above its water levels. As might be expected the contamination of the raw waters in the Bow River is many times that in the Elbow River. Examinations by the City Bacteriologist of these waters during twelve months, ending June 30, 1929, showed that, almost without exception, the raw waters of the Bow River indicate contaminations, presumably of intestinal origin, which might be either human or animal, while only one-fourth of the samples taken at the Elbow intake showed the same kind of contamination.

Experience in the past indicated that both river waters, in order to be regarded safe by the Public Health authorities, had to be too heavily chlorinated for palatability and both at times were turbid and otherwise unsatisfactory. Examinations showed that this applied in a greater or less degree at any point in these rivers within 38 miles of Calgary. It was desirable that such turbid waters should be filtered and while a satisfactory filtered water could be secured from the Bow River from any point above Calgary the filtration process would be simpler, and the results obtained would be better, from the Elbow than the Bow. At the same time the higher up the water could be taken from both rivers the better would be the quality of the raw water. It was found that filtration would be needed in any case and therefore the best scheme was that which would secure with certainty the required results at the least cost to the City. In order to accomplish this we saw no advantage in going long distances solely for the purpose of securing a better quality of water.

The foregoing argument led, therefore, to these conclusions:

1. The sources of supply had to be taken either from the Bow River or its tributary the Elbow River.

2. Considerable storage of water had to be provided both for the purpose of preventing the formation of frazil ice as well as to settle out and store indefinitely the deposited materials carried by the flood waters.

3. At no point on these rivers, no matter how far distant upstream from Calgary, could satisfactory water be secured without filtration.

4. In order to secure water at sufficient elevation for the supply of the whole of the city, distances measured along the pipe lines of 30 miles or more were necessary on the Elbow River and much greater

distances along the Bow River. This could only be done at great expense.

5. From almost every angle the water of the Elbow River was preferable to that of the Bow River.

6. Prior rights to the waters of these rivers either for power purposes or irrigation did not require to be seriously considered with intakes less than 38 miles measured along the aqueducts from Calgary.

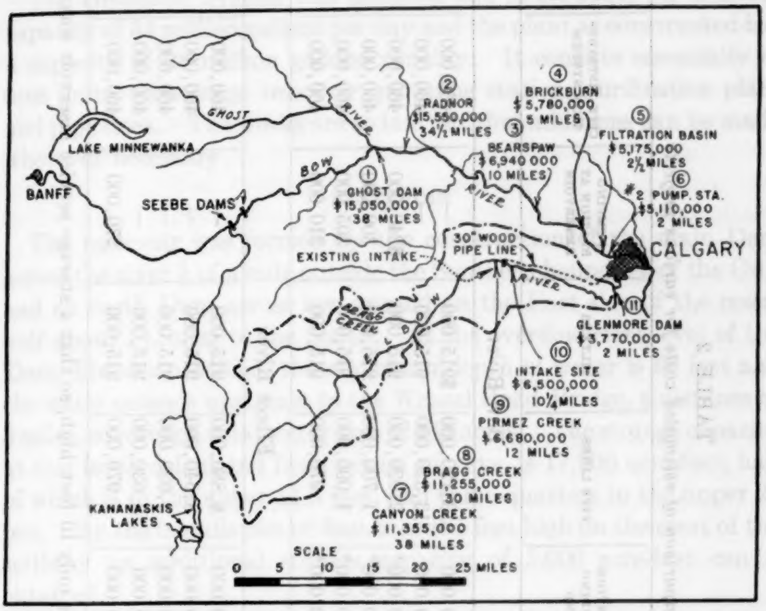

FIG. 1. LOCATION AND COST OF ELEVEN SCHEMES FOR EXTENSION OF THE CALGARY WATERWORKS

Elbow River watershed — west of Bragg Creek = 275 square miles, east of Bragg Creek = 140 square miles.

SCHEMES CONSIDERED

In all, 11 schemes were considered; 6 of them having the Bow River as the source of supply and 5 the Elbow River. These sources covered a great variety of possibilities, from schemes within the existing city boundaries to one scheme as far as 38 miles away from the city (figure 1).

A comparison of the estimated costs of the various schemes considered is given in table 2.

TABLE 2

Comparison of estimated costs for various projects

SCHEME NUMBER AND LOCATION	RESERVOIR INCLUDING LAND	PIPE LINES	FILTERS	PUMPING STATION AT RESERVOIR	CITY DISTRIBUTION SYSTEM	OVERHEAD TANK, HIGH LEVEL PUMPING STATION, REPAIRS TO SERVICE RESERVOIR, ETC.	ESTIMATED TOTAL COST
Bow River							
1. Ghost River	$400,000	$13,120,000	$915,000	—	$460,000	$155,000	$15,050,000
2. Radnor Power Dam	700,000	13,320,000	915,000	—	460,000	155,000	15,550,000
3. Near Alberta Ice Co. Ponds	2,250,000*	2,920,000	915,000	$240,000	460,000	155,000	6,940,000
4. Near Brickburn	2,250,000*	1,700,000	915,000	300,000	460,000	155,000	5,780,000
5. Infiltration Basins	2,250,000*	1,090,000	915,000	305,000	460,000	155,000	5,175,000
6. Pumping Station No. 2	2,300,000*	970,000	915,000	310,000	460,000	155,000	5,110,000
Elbow River							
7. Canyon Creek	960,000	8,880,000	915,000	—	460,000	120,000	11,335,000
8. Near Bragg Creek	1,120,000	8,640,000	915,000	—	460,000	120,000	11,255,000
9. Pirmez Creek	1,230,000	3,920,000	915,000	—	460,000	155,000	6,680,000
10. Existing Gravity Site	1,340,000	3,630,000	915,000	—	460,000	155,000	6,500,000
11. Glenmore	1,000,000	970,000	915,000	270,000	460,000	155,000	3,770,000

* The dam in each of these cases was only 10 feet high, so that a storage service reservoir was necessary for each and such was allowed for in the estimates.

The scheme recommended by the Consulting Engineers was No. 11, using the Elbow River as a source of supply, the reservoir to be located at Chinook, which was afterwards named "Glenmore Reservoir." The new works were placed under construction in August, 1930 and put in operation in January, 1933. It is the purpose of this paper to describe the various engineering features connected with the Glenmore Water Works System.

The Glenmore System was designed and laid out for an ultimate capacity of 84 million gallons per day and the plant as constructed has a capacity of 28 million gallons per day. It consists essentially of four units,—a storage reservoir, pumping station, purification plant and pipe lines. The works are so laid out that additions can be made wherever necessary.

RESERVOIR

The reservoir was formed by the construction of the main Dam across the river ⅝ of a mile outside the Southern boundary of the City and an earth Dam across low ground on the East side of the reservoir about 1½ miles to the South. At the overflow crest level of the Dam, Elevation 3492.5, the maximum depth of water is 60 feet and the water extends upstream to the Weasel Head Bridge, a distance of 3 miles, covering a total water area of 948 acres. The storage capacity at this level, calculated from actual surveys, is 17,900 acre-feet, half of which is in the upper 11.3 feet, and three-quarters in the upper 20 feet. By the installation of flashboards 5 feet high on the crest of the spillway an additional storage capacity of 5,000 acre-feet can be obtained.

In order to provide flattening out of the banks of the reservoir and to maintain control of land developments it was recommended that an area of 1,800 acres be purchased. To secure the necessary lands whole parcels had to be acquired in some cases, making a total area actually purchased of about 2,400 acres.

The low lying lands forming the bed of the reservoir were quite heavily wooded for this part of the country. The trees were cut off near the ground level and the brush and other vegetable growth up to a height of 2½ feet above the overflow level were removed from the site.

DAM

While the width of the valley at normal water level is 790 feet the overall length of the Dam itself is 1,050 feet, of which 441 feet is an

overflow with its crest 60 feet above the river bed. The Dam, which is of the straight gravity type constructed of Portland cement concrete, has a normal base width of 70 feet throughout the spillway portion. The bucket is continued out to form a pool 70 feet wide which contains a maximum of 6 feet of water below the outlet sill. The wall along the outer edge of the pool has a total height of about 41 feet and directs the flow of the water to the outlet channel, which has a width of 180 feet. Through the Dam near the base a tunnel is constructed from end to end for drainage, pipe lines and inspection purposes.

Over the top of the Dam is constructed a traffic bridge 27.5 feet above the crest and consisting of eleven reinforced concrete arches carried on piers spaced 70 feet apart along with approaches with walls and beam and slab construction. The roadway is just of sufficient width to accomodate two lines of motor cars with a 3 foot walkway on either side.

The foundation as disclosed by diamond drill exploration during the early part of 1930, and subsequently by actual excavation, was composed of horizontal layers of sandstone and shale free from any evidence of recent geological disturbance. The upper layer of sandstone was rather soft and broken and the shale was somewhat soft. The second sandstone layer on which the body of the Dam was constructed was hard with small seams and the underlying shale was firm and comparatively watertight.

DESIGN

The bridge was designed to carry a live load of 100 pounds per square foot on the whole or half span, 3,000 pounds on any square foot, or a 15 ton truck. Allowance was made for a temperature drop of 60 degrees and a rise of 50 degrees Fahrenheit. In the design of the Dam provision was made in the spillway for a discharge of 46,000 second-feet and in the pool and outlet channels a flow of 37,500 second-feet, being 3 times and 2½ times the maximum flow on record up to 1930. The lines of resistance were calculated for various conditions of loading as follows:

Case 1—Reservoir empty.

Case 2—Water level 9 feet above the crest.

Case 3—Water level at the crest with an ice thrust of 20,000 pounds per lineal foot.

The area of the cross section of the Dam proper is 3,000 square feet and the pool 640 square feet, increasing to 840 square feet at the outlet. For its height this Dam is one of the heaviest that has been constructed. Situated as it is immediately upstream from a densely populated portion of the City the structure was designed not only from the standpoint of absolute safety but to create and maintain a feeling of security in the minds of the residents.

The total weight of the Dam is more than 2½ times the combined lateral pressures of the water and ice (Case 3) and the friction on the foundations should be sufficient resistance against sliding. To increase the safety factor, however, the cut-off wall was tied into the body of the Dam by reinforcing steel in order to develop a high resistance in shear and the pool wall not only was keyed 10 feet into the rock but had behind it an earth embankment 25 feet in height. At the outer end of the discharge channel the floor was keyed 12 feet into the rock and rock blocks were laid on the bed of the channel for a distance of 40 feet downstream to protect this part of the work from erosion. Across the pool in front of the outlet channel at a short distance from the construction joint between the pool and the Dam an additional key about 5 feet deep was made. A large ratio of weight to pressure along with the great strength in the floor of the pool and the keys give the structure a very great resistance against sliding.

The stability of the Dam received a very severe test during the winter of 1932–33 when with the reservoir filling the ice adhered to the face of the Dam and was built up to a depth of 6 feet, creating not only a heavy thrust but considerable overturning moments as well. In the normal operation of the plant the water level in the reservoir will be continuously falling during the winter season as the storage will have to be drawn on to meet the demands of the pumping equipment.

Drainage and grouting

In order to prevent or reduce the possibility of water flowing underneath the structure a cut-off wall about 10 feet wide was constructed at the heel of the Dam to a depth of several feet below the lowest pore or seam through which water was leaking, or to a maximum depth of 30 feet below the bed of the river. Drainage pipes 2 inches in diameter and 5 feet apart were placed to connect the foundation under this cut-off wall to the drainage channel in the tunnel in order to

relieve possible uplift pressure. At certain intervals holes were drilled below these pipes to a maximum depth of 20 feet and it was intended that should any of these pipes indicate the presence of water underneath, passages in the rock would be sealed by pressure grouting with cement. A few additional pipes were placed on the downstream side of the tunnel to relieve any pressure which might accumulate under the body of the Dam.

Grout pipes 2 inches in diameter were connected to the seams in the sandstone layers through small boxes and carried either into the drainage tunnel or through the concrete and after a sufficient layer of concrete had been placed and set these seams were sealed by pressure grouting. The grout was composed of Portland cement and water of such consistency that it would flow easily and penetrate the very small seams. The pressure was varied according to the requirements of each particular hole.

Vertical construction joints

The Dam and pool were divided by vertical construction joints into areas in general 70 feet square. These joints had numerous substantial keys and near the upstream face of the Dam a copper plate $\frac{1}{4}$ inch by 8 inches was installed as a water-stop. One-half the plate was embedded in the concrete when placed and the other half was painted with an asphaltic gum in order that it might slip in the concrete placed in the adjoining section. In order to prevent bond the construction joints were covered with asphaltic paint.

Concrete construction

There were two classes of concrete specified, Class A of 3,000 pounds and Class B of 1,500 pounds per square inch at 28 days standard curing. Class A was used for the bridge, the cut-off wall and a 12 inch facing on the Dam, including the portions in contact with rock. Class B was used throughout in the interior of the Dam.

Special attention was paid to the bonding of the concrete to the rock or to concrete previously placed. The rock surface or the aggregate in the concrete was cleaned by wire brushing and scraping followed by washing thoroughly by a hose stream, care being taken to remove all loose particles. Immediately before beginning the placing of the concrete the surface was again cleaned and wetted by water hose and a layer of mortar one inch thick placed on the surface and spread and worked by brooms.

The concrete was placed in 5 foot lifts, being deposited in layers 12 inches in thickness. Each layer was thoroughly tamped by the use of a mechanical vibrator. In finishing each lift the surface was sloped upwards 1 foot in a downstream direction and left fairly irregular. Several keys about 12 inches deep were formed to increase the resistance against horizontal shearing stresses. That these precautions against leakage through horizontal joints have been effective is demonstrated by the fact that no leakage whatever has been detected through them and it is quite probable that the uplift assumed in the design at the upstream face of the Dam does not exist.

Each concrete section immediately after being placed was protected from extreme temperature and was kept wet for a period of 14 days. In carrying on concrete work in freezing temperatures the aggregates and water were heated by steam and immediately after placing in the structure the section was enclosed in canvas and a minimum temperature of 45 degrees Fahrenheit was maintained for a period of at least three days.

As a result of the precautions taken during construction, tests made in 1933 showed the total drainage or leakage into the tunnel from all sources was only 3 gallons per minute, with the result that no grouting of the foundation at the bottom of the cut-off wall or elsewhere was necessary.

EARTH DAM

The earth Dam constructed across a low-lying area on the East side of the reservoir is over 3,000 feet in length with a maximum height of 22 feet and its top is 15 feet above the crest of the main Dam. The top width is 15 feet with a 3 to 1 slope on the water side and 2 to 1 at the back. The portion of the slope in contact with water is paved with rock.

The top soil was removed from the area on which the Dam was placed and a cut-off trench excavated in the middle. The fill material was procured near the site and consisted of sandy clay. It was hauled to the job in large caterpillar wagons by gasoline tractors and deposited in layers. The traffic of the heavy equipment over the fill was very effective in bringing about consolidation. The volume of material used, as measured in the borrow pit, was 129,000 cubic yards, which was about 3 per cent greater than the actual volume of the earth dam.

Flood control

Like most mountain streams, and as already stated, the Elbow River is subject to flood flows resulting in considerable damage being done to properties in the low-lying parts of the City. Prior to the design of the Dam the largest flow on record was 15,300 second-feet, which occurred in 1929 and caused considerable damage along the route of the river. At the end of 1931 the City instructed the Engineers to install in the Dam through the passages provided for the flow of the river during construction, conduits and valves of such capacity as to make use of the storage reservoir to reduce the flood flows downstream. This flood control equipment, which was installed in 1932, has a discharge capacity of about 9,000 second-feet and under normal conditions the water level in the reservoir can be quickly lowered to provide storage for the excess flow. While the human element enters into the operation of this system it is intended that, previous to the flood period, which is usually early in June, the water level should be lowered as far below the crest as will permit of the satisfactory operation of the pumping equipment.

The early June domestic consumption of 20 million gallons per day can be met with the water level about 15 feet below the crest as the electric power is always available to meet a sudden increase in load. At this level the storage is only 6,800 acre-feet and this can be further reduced on information 6 hours in advance from the gauging station observer at Bragg Creek 30 miles up the river. With proper operation of the flood gates the river flow of 26,000 second-feet into the reservoir may be reduced to a flow of 13,000 second-feet downstream.

Before this work was installed the principle of this system of control received a very severe test when in the first three days in June, 1932, following a rainfall of 3.13 inches in 24 hours, one-seventh of the whole year's precipitation, the Elbow River experienced the highest flood flow in its history. The flow increased from 2,600 second-feet at 6 P.M. on June 1 to a maximum at 10.30 P.M. on June 2 of 26,000 second-feet or 70 per cent higher than any previous flow. Only one opening through the Dam was available and its capacity was such that the reservoir filled up to a height of 18 inches below the crest by 9 A.M. on June 3. The maximum flow downstream of 12,000 second-feet, 46 per cent of the total river flow above the Dam, was sufficient to flood a few cellars but one can only imagine what the damage would have been had not the Dam curtailed the flow.

PUMPING STATION

The Pumping Station, which has been constructed on the unit of the Dam through which were the three passages for carrying the river flow during the construction of the Southern portion, includes not only the pumping equipment for the raw and filtered water but also the intakes, gates, screens, conduits, etc., in connection therewith.

Pumping Equipment

A study of the river flow records for the previous 18 years, and the estimated demand for water for domestic consumption and for water power for pumping, indicated that while the installation of hydraulic turbines would do most of the pumping an auxiliary plant must be installed to operate during periods of low flow in the river. Based on a domestic supply of 18 million gallons per day throughout the winter months the storage provided was found to be sufficient to operate the turbines throughout 7 of the 18 years but auxiliary power would have to be used on an average of 13 days per year. With a domestic water demand of 27 million gallons per day during the winter auxiliary power would have to be used every year and for an average of 64 days. The value of the available water power at current rates for electricity would be about $43,000 per annum for an average rate of 18 million gallons per day, increasing to $55,000 per annum for an average rate of domestic supply of 27 million gallons per day.

There was never any doubt of the economy of using water turbines as the primary power supply, but the selection of the type of stand by service was made only after a careful study of the capital and annual cost of installations of Diesel engines, steam turbines and electric motors. Undoubtedly the conditions were favorable to the installation of Diesel engines as the plant would be independent of outside sources for all its power. With the usual charges for electric standby service the annual cost of the Diesel installation was the lowest, but taking into consideration the fact that even with low water the turbines could be operated for short periods to avoid increase in peak loads, or in case of failure or shortage of electric power, the officials of the City's Electrical Department decided that the service could be given to the water works at a price which would make the annual cost of the electric standby service compare very favorably with that of the Diesel engine.

The Pumping Station has a normal operating capacity of 28 million

gallons per day of raw water but only 22 million gallons per day of filtered water as it is intended that about 6 million gallons per day will be supplied through the Eastern pipe line to the low-lying areas on the East side of the City by gravity direct from the Water Purification Plant reservoir.

The pumping equipment as installed consists of five units with an aggregate capacity of 94 million gallons per day, three units being for the pumping of raw water to the Purification Plant and two for pumping the filtered water to the City.

The electric power for operating the motors is brought in on overhead wires from a near-by substation at 12,000 volts. A circuit breaker with a rupturing capacity of 600,000 kv-a is installed in the outdoor substation at the North end of the Pumping Station. Three 275 kv-a transformers reduce the voltage to 2200 and the power is carried to the switch panels through underground lead covered cables laid in steel conduits. The substation also contains the three—25 kv-a, 12,000—220 volt transformers for operating the small motors around the plant as well as for indicators and lighting. Single pole knife switches are installed on the primaries of the power transformers so that they can be disconnected when not in use and when there is no danger from condensation.

Inlet at dam and conduits for domestic supply

The water for domestic supply is drawn through ports at the upstream face of the Dam extending down to within 22 feet of the bottom of the reservoir. The water flows through two passages 6 feet wide, in each of which is placed a bar screen 6 feet by 5 feet and a motor driven rotating screen 6 feet wide extending up 8 feet above the top of the Dam, a total of 55 feet. The bar screens have openings of 4 inches wide and are installed in steel lined grooves with stop logs extending above the water level so that the depth at which the water is brought in may be varied to suit the conditions.

The travelling screens were manufactured by the Link Belt Company in Toronto and have numerous panels and openings $\frac{1}{4}$-inch square in the bronze wire mesh and all debris is washed off the screen into a trough at the back by a spray of water.

After passing through the screens, in which any weeds or other debris are removed, the water passes through sluice gates, down a shaft and through a 5 foot diameter steel pipe into the small turbine or the header of the raw water pumps. It is then pumped through

a 48-inch concrete pipe with three 36-inch branches into the coagulation tanks in the water Purification Plant. From the clear well of the water purification plant the water returns through a 48-inch concrete pipe. Some of the water passes through a branch into the eastern pipe line and flows directly to the low pressure area but the major portion portion flows into the header of the filtered water pumps and is pumped to the City's distribution system through the western 36-inch pipe line. Valves are provided, however, whereby the water may be pumped to the City by both or either of the two pipe lines.

Conduits and gates for flood control and power

The turbines are placed in the central of the three temporary passages through the Dam and the two larger turbines take their supply of water through butterfly valves and 5-foot branches off the flood control conduits or penstocks which are placed in the outer passages. Through the Dam each conduit is 10 feet wide by 16 feet 6 inches high, semi-circular top and bottom, and changes to a circular section of 12 feet diameter at the downstream end on which a butterfly or pivot valve is installed. On the upper end the water for each conduit enters through two ports, each 6 feet by 20 feet, in which are placed the trash racks, or if desired to shut off the water at the upstream face the trash racks can be removed and replaced by roller gates. The racks are made up of structural sections with bars spaced 3 inches apart.

The gates are made up of a steel skin plate and structural sections with cast steel rollers rotating on fixed stainless steel pins. The roller paths are made of structural steel embedded in the concrete. The frame of one of the gates is filled with concrete to give it additional weight to ensure closure with the downstream valves open. In operation the light gate is placed first and removed first, thus it is not necessary to lift the heavy gate with pressure against it. Under normal conditions the pressures on the two sides of the gates are practically equalized by the installation of a 12-inch diameter cross-over pipe and valves with a vertical riser that acts not only as a pressure equalizer but also as a vent in emptying or filling the conduit.

The two 12-foot diameter pivot valves used for flood control purposes are of the Dow disc arm type with horizontal spindles at Elevation 3440.0. The cast steel casings are riveted to the steel conduits. The hoists or headstocks, which are arranged for manual operation, are located on the floor 18.5 feet above and 4 feet downstream from

the centre lines of the valve spindles on the vertical centre line of the conduit. The hoist operates the disc through a heavy screw crosshead and connecting rod attached to the disc near the bottom.

The hoist has reduction gearing such that the valves may be completely opened or closed in about two hours. In the full open position the disc is held tightly against a stop which eliminates all vibration, even under the severe operating condition of free discharge of water under a head of over 52 feet. When fully opened the disc is balanced but when closed a few degrees there is a tendency to close, which reaches a maximum when the valve is 15 degrees from the horizontal. This changes to a tendency to open when the disc approaches the seat. In the closed position the top of the disc is held against its babbit seat by the water pressure and the bottom is held in by the hoist, with the result that the leakage is small. The whole valve is encased in concrete with manholes for access to the jacks for removing or adjusting the trunions.

Meters and indicators

For measuring and recording the quantity of water pumped three Venturi meters are installed on the raw water, filtered water and the eastern mains respectively. The upstream rings and cones are of cast iron but the throats are of cast iron with bronze liners. The downstream cones are made of concrete pipe similar to the mains in which they were installed. Joints are made by using Victualic couplings.

The raw water meter is 48 inches in diameter with a 30-inch diameter throat and has an indicating and recording unit placed in the alum machine room of the Chemical Building. The filtered water meter tube is the same as that of the raw water but the float chamber and indicator are located in an underground meter chamber and the movements are transmitted electrically by a "Selsyn" unit to the indicating and recording instruments in the chlorine machine room in the Chemical Building.

The readings of both meters are transmitted electrically to 24-inch diamater indicators situated on the eastern wall of the Pumping Station and the filtered water meter has an additional indicator on the end of the Filter Operating Gallery.

The 36-inch meter tube on the eastern main is situated near the Pumping Station and has a throat 22 inches in diameter. The indicating and recording instrument is placed on the floor of the Pumping Station.

Two additional "Selsyn" indicators reading the water levels on the filters and in the service reservoir on 24th Street are also placed on the wall of the Pumping Station beside those of the Venturi meters. With these four indicators the operation of the plant is very simple and no automatic apparatus for controlling either water levels or pressure is necessary.

Pump tests

In selecting the pumping equipment special attention was paid to efficiency and the contract provided for liquidated damages to the extent of, for the large units, $1,500.00 for each 1 per cent that the actual efficiency determined by the tests made after installation was below those guaranteed by the contractor. In the testing of the pumping equipment three different systems were used.

In order to obtain the desired accuracy all instruments were carefully calibrated both before and after the tests. Even the Venturi meter tubes were calibrated by volumetric measurements in one of the settling tanks, which had been measured.

The tests of the motor driven pumps were quite easily made and consisted of direct measurements of the electrical input and hydraulic output. The input power meter equipment consisted of a polyphase watt meter, an ammeter, a voltmeter and instrument transformers both current and potential. In determining the output power of each pump the pressures on both the inlet and outlet were measured by gauges installed in duplicate, while the discharges of the pump was read by a Venturi meter on which was installed a water differential manometer, the indicator being used only for checking purposes.

The contract called for each unit to have a capacity of 22 million gallons per day against a total lift of 90 feet and the contractor guaranteed under those conditions that the efficiency would be 83.1 per cent and that with a total lift of 95 feet each unit would discharge 19.7 million gallons per day with an efficiency of 83.1 per cent. The tests showed that both the capacity and efficiency guarantees were exceeded, the capacity being 20.6 million gallons per day and 22.5 million gallons per day for the 95 foot and 90 foot heads respectively and the efficiency obtained under the specified conditions was 84.6 per cent.

In the testing of the turbine driven pumps no direct means was available for measuring the water passing through the turbines and we desired to avoid the cost of installing a weir or other measuring device for this purpose. Conditions were such that accurate current

meter measurements were difficult to make and after careful consideration of all the known methods of making these tests it was decided to use the Salt Titration Method. In this method a strong, almost concentrated, solution of salt was pumped into the inlet of the turbine at a certain definite rate. The discharge from the turbine was sampled downstream in the discharge channel. The discharge was computed by multiplying the rate at which the salt solution was fed into the turbines by the ratio of the amount of salt in unit volumes of the primary and secondary solutions.

The solution of ordinary salt in water was made in a mixing tank and transferred to a supply tank carefully calibrated for volume between certain points in order to maintain a constant rate of flow. The solution was then fed into an orifice tank in which the head was kept constant, thence into a pump suction tank where water was added to again maintain a constant level and discharged into the turbine inlet pipe through a perforated pipe across the diameter. The actual rate of solution flow was determined by timing with stop watch the fall in the level in the supply tank. Samples of the solution were taken from the mixing tank and the turbine discharge was sampled through a perforated pipe grid placed in the outlet channel. Water was drawn through the grid continuously and a portion sufficient to fill the sample bottle in 30 seconds was diverted.

As the salt tests were rather cumbersome they were used only to secure a few points at the conditions which were guaranteed and to calibrate a differential water manometer which was installed to read the loss in pressure between the reservoir and the inlet to the turbines. A coefficient was obtained from the salt tests which showed that the discharge of the turbines could be read directly from this manometer.

For measuring the pressure head on the turbines the pipe was connected at four points on the inlet pipe and the water level was read direct in a glass gauge carried up the side of the Dam. The tail water level was read by a float in a stilling chamber. Corrections were made for the velocity head in the inlet pipe and in the discharge end of the draft tube. The output of the pumps was measured in the same way as for the motor driven pumps. As both ends of the shaft were inaccessible the speed was determined by measuring the peripheral speed of the shaft by a revolution counter with traction wheel specially calibrated. Under the specified conditions the overall efficiency of the raw water and filtered water pumping units was 78.7 percent, which was slightly above that guaranteed.

In the test of the small water turbine driven pump the Salt Titration Method of test was considered to be rather unsatisfactory owing to the position of the pump in the discharge chamber and the difficulty of obtaining representative samples of the discharge due to dilution. The water for operating this turbine is taken off the pipe to which the raw water pumps are connected and in order to obtain the discharge of the unit all but one of the six sluice gates on the downstream side of the revolving screens were closed and this one was partially closed to form an orifice with a differential which could be easily read. This orifice was calibrated in the normal operation of the other raw water pumps as all the water drawn through it passed through the raw water Venturi meter and this orifice was used to measure the amount of water passing through the small unit. As the discharge of the pump was measured by a Venturi meter the amount of water required for the turbine was the total amount passing through the sluice gate orifice less the amount passing through the pump.

During the tests the head on the turbine was above the normal head of 53 feet and corrections were made in the head and discharge of the pump to reduce them to the basis of the normal turbine head, it being assumed that the efficiency remained constant.

WATER PURIFICATION PLANT

The Water Purification Plant, which is situated on the high ground near the North end of the Dam, has its filter water level 72 feet above the crest of the Dam and has a filtering capacity of 28 million gallons per day at standard rates or a net capacity of 27 million gallons per day after allowing 1 million gallons per day for wash water and other plant service. It is so designed that it may be triplicated with a minimum of interference to its operation.

The purification process consists essentially of four stages,—coagulation with alum, sedimentation, sand filtration and sterilization of the effluent with chlorine.

The filter alum, aluminium sulphate, is stored in large bins holding 100 tons in the Chemical Building and is fed into the raw water in solution by two "Omega" alum feed machines of the dry feed type. These machines are provided with weighing hoppers and adjustable feed mechanisms which discharge the alum into lead lined chambers where it is thoroughly agitated and mixed with water and the solution is then fed into the raw water main by water ejectors at a point immediately downstream from the throat of the Venturi meter. After

passing through about 100 feet of pipe the treated water enters the three coagulating units through branches especially designed to equalize the flow. Each of the three units of coagulating or mixing tanks consists of four chambers 20 feet square and 30 feet deep. The water flows through these tanks in series with the inlet and outlet ports at the side walls alternately at the bottom and at the top, which produces a gentle spiral flow for a period of 45 minutes at normal rates. This mixing action is very gentle and brings the particles of alum, sediment or bacteria into intimate contact, forming particles of "floc" of such size and density that much of it can be settled out. After passing through the fourth stage the water flows into a distribution channel and thence passes into settling tanks through four diffuser units designed to give a very even distribution of the water throughout the upper half of the tank.

The three sedimentation tanks are each 40 feet wide, 237 feet in length and 23 feet deep, which at the normal rate of 28 million gallons per day provides for a settling period of $3\frac{1}{4}$ hours. The water after passing through these tanks discharges over submerged weirs and through 4 foot square sluice gates into the settled water conduit in the Filter Plant. The floors of these tanks slope about 1 foot towards the discharge end where there is a wash-out cross-channel with a sluice gate connected to the main drainage channel. When it becomes necessary to wash out the sludge which has accumulated on the bottom of the tank the water is drained out through the drain valve and carries considerable sludge with it. The inlet valve is then partially opened up for short periods, causing a flow down the sloping bottom of the tank and washing out the sludge. The floors, walls and columns are then washed clean by means of a hose, pressure piping having been installed for this purpose. The coagulation or mixing tanks due to the movement of the water retain very little sediment but for drainage purposes a 12 inch mushroom valve is placed in the bottom of each chamber.

The Filter Plant is made up of eight units, each with a sand area of 1,456 square feet, which at normal rates of filtration provides a capacity of 3.5 million gallons per day or a total plant capacity of 28 million gallons per day before deducting wash water. The filters which are arranged on both sides of the central operating gallery are 26 feet wide along the gallery and 56 feet long and 9 feet 6 inches deep. Along one side of each is a covered channel 4 feet wide which carries the settled water into, and wash water out of, the filter. The filter

tanks are constructed over the top of the filtered water reservoir, which has a depth of 10 feet and a capacity of 800,000 gallons.

At normal operation of the plant the settled water comes in from the settling tanks through a conduit, which is constructed along the outer ends of the filters and across one end of the plant, thence through square sluice gates into the side channels, then into the filter tanks through the wash water trough openings, down through the filter medium of sand and gravel, through the underdrainage pipes into a large circular conduit constructed of concrete on the underside of the filter floor and then through rate controllers and piping into the reservoir below.

When the resistance through the filter sand increases to 8 or 9 feet through the extraction of the dirt from the water the filter is washed by a reverse flow. The wash water is taken from the storage tanks provided in the upper floor of the Administration Building through large piping and valves provided therefor, the dirty water overflowing into the wash water troughs and through the drainage system to the river.

The filter medium consists of 6 layers of gravel and sand. The gravel is made up of 4 layers. The layer which surrounds the underdrain laterals has a depth of 8 inches, consisting of particles passing a $1\frac{1}{2}$ inch standard A. S. T. M. sieve but retained on one with $\frac{3}{4}$ inch openings. The second, third and fourth layers are each 4 inches in thickness with particles progressively decreasing in size, being $\frac{1}{4}$ to $\frac{1}{8}$ inches in the fourth layer. Above this gravel is a 2 inch layer of coarse sand ranging in size from a No. 8 to No. 20 sieve. On top of this is 28 inches of filter sand, which has an effective size of 0.5 millimeters and a uniformity coefficient of 1.4.

The gravel, which was taken from the Bow River, consists of limestone which was thoroughly washed and carefully screened and placed in the filters. The sand consists of smooth round particles of which 93 percent was silica and was obtained from the beach of a small lake about 40 miles North-east of Calgary. The sand was screened at the lake to remove the gravel and foreign matter and was prepared at the plant by means of an hydraulic separator designed on the principle of the filter back wash system. The fine particles were floated over by the high upward current of water and the coarse sand was separated from the regular filter sand by a vane just above the bottom and the two grades were extracted through separate compartments. The coarse sand was stored and periodically placed in the filters, while

the regular filter grade flowed into a hopper and was discharged into the filter by a water ejector. With frequent tests and constant supervision the velocities of the water in various portions were regulated to obtain quite an accurate separation with a minimum loss of sand.

The underdrainage system consists of 4-inch cast iron pipes placed at 12-inch centres crosswise of the filters connected through 6-inch tees to the 48-inch diameter concrete manifold underneath the filter floor. Brass nozzles $\frac{13}{16}$-inch in diameter are placed in the bottom of the laterals at 12-inch centres so that there is an opening in the centre of every square foot of filter area.

In the operation of the filters the rate of filtration is controlled by a manual master control either alone or in conjunction with an automatic float control operating on the water level in the reservoir below. When the reservoir water level is 12 inches or more below the overflow the filter rate is regulated by the manual control only. But as the water level rises above this limit the float control comes into action, decreasing or increasing the rate of filtration of all filters according to the rise or fall of the water level, and reducing the rate to zero as the water reaches the overflow level. By these controls the operation of the plant is very convenient and requires very little attention. If the rate of pumpage to the City is increased the water level in the reservoir is immediately lowered, which increases the filter rate, thus lowering the water level on the filters. This is seen on the indicator in the Pumping Station and the raw water rate of pumpage is increased to keep the level constant. The float control also has a decided advantage during back washing, because when one filter is taken out of operation the rates of the remaining filters are increased to maintain a constant output, thus avoiding the changes in the water levels or in the pumping rates. In the operation of the plant the manual control is set at the normal capacity of the filters and all regulation is done by the float control.

The two wash water tanks are of steel construction 31 feet 6 inches in diameter by 8 feet deep and obtain their supply from the western 36 inch pipe line through a 16 inch cast iron pipe. An hydraulic valve on this line is opened or closed by a float in the wash water tank and is regulated by a Venturi rate controller with meter indicator recorder. In operation this controller is opened when the water level in the wash water tanks falls 1 foot and remains open until the tanks are filled up again after the filter wash.

The substructure throughout is of Class A concrete with a specified

strength of 3,000 pounds per square inch at 28 days and is reinforced with medium grade steel.

The structural members of the superstructure are reinforced concrete but the walls are constructed of red brick with Tindall stone used for base courses, parapet and trim. The roofs over the settling tanks and Filter House are of asphalt and gravel laid on cork insulation, while a copper covering was used on the pitched roof of the Administration Building.

The Chemical Building houses the bins and machines for feeding the alum into the water, the chlorine machines and the storage for the chlorine cylinders and the machines for sterilization and the heating plant. This building is of similar construction to that of the Filter Plant.

PIPE LINES

The pipe lines consist of one 48-inch main about 1,200 feet long carrying the raw water from the Pumping Station to the Water Purification Plant, one 48-inch main about 1,300 feet long carrying the filtered water from the pure water reservoir back to the Pumping Station, a 36-inch main 16,500 feet long in the western section connecting to the existing 24-inch steel main, a 36-inch main 4,230 feet long connecting the western section to the service reservoirs and a 36-inch main 20,800 feet in length from the Pumping Station through the eastern section connecting to the existing 24-inch steel main, a distance of 2¾ miles from the connection to the western main.

These pipe lines were made of what is known as the steel cylinder reinforced concrete pipe. The 36-inch pipe, made in laying lengths of 12 feet, consisted of welded steel cylinders No. 14 B. and S. gauge blue annealed sheets with galvanized steel bell and spigot joint rings welded to the ends. The interior was lined with 1½ inches of concrete and surrounded on the outside by concrete to a thickness of 2½ inches reinforced by welded steel hoops. The pipe was so designed and placed that the stress in the steel due to the internal pressure of the water was less than 12,000 pounds per square inch after adding to the static head the increase in pressure due to friction or other causes. All the pipe was manufactured within the City, the materials being obtained in Alberta with the exception of the steel plates which were manufactured in Ontario and the rolled steel joint rings which were imported.

The 48-inch pipe is similar to the 36-inch except that the lining was

reinforced by steel mesh tack-welded to the steel cylinder and the total thickness of the pipe is $4\frac{1}{2}$ inches. The steel cylinder was made of No. 12 gauge.

The manufacture of the pipe was started early in January 1931 and the whole work was completed and tested before the end of the year.

The steel plates were rolled to shape and both longitudinal seams and circumferential seams connecting to the joint rings were electric arc welded. Bulkheads were then placed on the ends of each length and the pipe was subjected to an internal air pressure of 5 pounds per square inch to detect any leaks.

The reinforcing steel was made into a cage consisting of a few $\frac{1}{2}$-inch round longitudinal bars to which was welded the circular bands. The whole unit was placed as a unit over the steel shell and the longitudinal bars were welded to the bell joint ring. The circular bands were made from mild steel flat bars of sizes varying according to the pressure requirements. The bars as they came in from the mill were cut to the proper length, rolled to shape and the ends welded by the electric fusion welding process and then each bar was placed in a testing machine and the weld was stressed to 24,000 pounds per square inch. The steel cylinder with the reinforced cage attached was then placed in steel moulds with bell end down and the concrete poured around it. The pipe was cured for a period of $2\frac{1}{2}$ days in steam under a temperature of 100 degrees Fahrenheit. At the end of this time the pipe was removed to the storage yard, from where, after a further period of four or more days, it was hauled to the work and placed in the trench.

The quality of the concrete was the most important feature of this work. We have all heard construction men complaining about the difficulties of placing concrete in thin reinforced walls, meaning thereby walls anywhere from 10 to 18 inches in thickness, but in this part of the work we had walls 12 feet high with thicknesses of $1\frac{1}{2}$ inches on the inside and $2\frac{1}{2}$ to 3 inches on the outside shell. Further the ordinary defects, such as honeycombing, which with patching might be acceptable in most concrete work, were not tolerated on the manufacture of this pipe and any length of pipe which showed on the removal of the forms any imperfect concrete was immediately rejected with no option of repair. In the first day's operation five lengths of pipe were rejected for this reason due to improper combination of the aggregates and to imperfect tamping, but during the manufacture of

the rest of the pipe the concrete work was almost perfect and only two or three lengths in all were rejected. The contractor therefore constructed the equivalent of a wall 12 feet high and nearly 15 miles long with thicknesses varying from $1\frac{1}{2}$ to 3 inches with scarcely a defect. With this experience one is forced to form the opinion that the numerous defects in ordinary concrete construction are wholly unwarranted.

The aggregates used in the making of the concrete were clean, well graded and of three sizes, as follows: Gravel with all particles passing a $\frac{3}{4}$-inch but retained on a No. 4 sieve, coarse sand having fineness modulus of about 3.5 and fine sand, which was mostly of silica, with fineness modulus of about 2.0.

These three aggregates were proportioned to make a well graded workable mixture, the fineness modulus being maintained at about 5.0. The average amount of cement used was about 10 bags per cubic yard of concrete, which gave an average strength at 28 days standard curing of 4,000 pounds per square inch, using a slump of about 9 inches. As the quality of the concrete turned out was well above the requirements of the specifications the proportions of the mix while set by weight were actually measured volumetrically in special wheelbarrows with adjustable boxes. The concrete was mixed for at least two minutes in three small mixers from which it was dumped into a storage hopper and thence into a conical bucket with a discharge valve in the bottom. This bucket was handled by a locomotive crane operating on a railway track. This crane was also used for handling the pipe, doors and roof of the curing bins, forms and other material. The steel forms were filled directly from the bucket and the concrete was hand spaded or tamped for a half hour or more to ensure density.

Samples of the aggregate were tested each day and test cylinders of the concrete were made daily and cured either under standard conditions in the laboratory or under exactly the same conditions as the pipe itself and were broken at either 7 or 28 days. Specimens of the welds in the circular hoops as well as in the steel cylinders were taken each day, machined and tested in the City's laboratory. In the reinforcing bands the average strength of the fusion weld in the 439 tests made was 56,400 pounds per square inch and in the plate the average strength of the arc weld in the 82 tests made was 45,600 pounds per square inch.

The laying of this pipe was carried on in much the same way as for the cast iron pipe except that the joints were caulked from the

inside. The gaskets were made from lead rings wedge shaped in cross section and fibre filled. In the laying of the main the gasket was placed in the bell of the pipe which had been laid and after several pipes had been placed in position the gasket was driven up into place but not tightly caulked. The space on the outside of the joint between the two lengths of pipe was then filled with a cement grout. After the laying had been completed, the backfilling in position for some length of time and when the pipe had an opportunity of settling into its final position the joints were finally caulked tight and the joint space on the inside of the pipe filled up with mortar to make a continuous smooth cylinder.

In the laying of a pipe of this size through City streets it was impossible to so lay out the profile that all obstructions would be avoided. While the depth of most sewers can easily be determined beforehand there are always a few drains, water mains or other pipes in which the available information is either incorrect or entirely lacking. To provide for changes in the profile there was always kept on hand a number of special pipes in which the spigot rings were mounted at certain angles with the centre lines, thus securing a deflection on each pipe. Where curves were designed either on the horizontal or vertical the deflection required for each pipe was computed and the spigot rings tapered to suit. Very small deflections up to $1\frac{1}{2}$ inches per 12 foot length were satisfactorily made with standard pipes.

Branches were made of steel flanged castings welded to the steel cylinder to which was also welded steel plates for reinforcing purposes. The section was then strongly reinforced by heavy steel bands and straight bars with the addition in some cases of structural steel. The interior was lined with concrete in the shop and thoroughly encased in concrete in the field after the joints were made.

At intervals of about 2,000 feet in the 36-inch main, 30-inch hand operated valves were installed and connected by Victaulic couplings to the cast steel rings provided on the ends of the special conical pipes.

The contract for this work required that the leakage throughout should not be greater than 667 Imperial gallons per 1,000 feet of pipe per day when tested under a pressure 50 percent greater than the normal operating pressure. After the laying and jointing work was entirely completed leakage tests were carried out to determine if the requirements of the contract had been fulfilled.

Sections of the mains with lengths varying from 2,328 feet to 8,101 feet were bulkheaded at the locations where the valves were to be

placed, the main filled with water and the pressure slowly built up to the test pressure for that particular section which was held constant by a pressure relief valve. The actual pressure in the main was measured by duplicate gauges placed on a line connected directly to the main. During the tests the water was pumped from a calibrated measuring tank directly into the main and the return through the relief valve flowed into the tank so that the actual leakage of the main was represented by the fall of the water level in the measuring tank. Hourly readings of the leakage were taken for a period of 24 hours. The results of these tests indicated that the leakage varied from a minimum of 28.61 to 63.3 gallons per 1,000 feet of pipe, the average being 45.6 gallons per day per 1,000 feet of pipe or only 6.8 percent of the leakage permitted under the contract. The total rate of leakage through the whole of the 36-inch pipe lines amounts to 1.3 gallons per minute.

The mains were not in service when the tests were made and each section was filled with water for only a few days before the tests were carried out. It is therefore not surprising to find that the actual leakage decreased continuously throughout the 24 hours, being 17 percent less for the last hour compared with the first.

The manufacture, laying and backfilling to the top of the pipe was carried out under contract with the Canada Lock Joint Pipe Company, but the excavation and backfilling of the trench was done by the City's relief labor.

The surface of the concrete on both the inside and outside of the pipe was exceptionally smooth throughout and it was expected that the friction loss would be low. Tests carried out six months after the pipe lines were placed on continuous use on a length of about 4,900 feet of 36 inch pipe showed an average coefficient of 156 in Williams and Hazen's Formula, which it is understood is the highest that has been obtained on a pipe of this size. The increased value to the City over a similar sized pipe with a coefficient of 100 may be realized by comparing the relative carrying capacities of approximately 28 million gallons per day and 18 million gallons per day respectively for a 3 foot loss of head per 1,000 feet of pipe.

ARCHITECTURAL TREATMENT

There was considerable opposition to the locating of the plant so near the City on the grounds that the Dam and associated works would be unsightly in appearance and mar the natural beauty of the

district. With this in mind special attention was given to the general layout of the plant and its architectural treatment.

The structure of the Dam is suggestive of strength and security and the design of the bridge gives a sense of scale to its sturdy proportions. The Pumping Station and Screen House have been considered as part of the general mass and the wall lines of the pool studied in relation to the original contours of the valley so that with the completion of the finished grading and the planting of suitable trees the whole scheme will merge naturally into the landscape.

The Chemical and Heating Building is logically placed and has been treated in a simple manner as a service unit built of reinforced concrete and faced with brick and stone trim. As the Administration and Filter Building has more of the character of a public building the treatment of brick and stone has been elaborated accordingly. In the selection of materials for the interior finish the questions of appearance, cleanliness and economy of maintenance were considered. At the entrance hall and main staircase the floors are of travertine and the walls of Notre Dame marble. The floors of filter gallery and walkways are terrazzo and the lower walls of the gallery and filter operating tables are tiled in mottled green. The bronze instrument cases on the tables and the recording dials and signals are also designed as part of the architectural scheme.

CONCRETE

The concrete on the whole work amounted to 120,000 cubic yards and the control of its quality was one of the most important duties of the field engineers. As a great deal of the concrete was in contact with water and some of it subject to severe frost action it was necessary that it should be dense, watertight and strong. Moreover the cement was paid for by the bag in all contracts except that for the pipe lines and constant supervision was necessary for economical reasons.

The largest volume of concrete placed on any one day was about 1200 cubic yards, of which 997 cubic yards were placed in the Dam. The maximum amount of concrete placed in any week was about 5800 cubic yards and the biggest month's work was in June 1931 when about 23,000 cubic yards were placed, 17,500 cubic yards of this amount being placed in the Dam.

Laboratory

To assist in the carrying out of the construction work a field laboratory was constructed as part of the field office and was used in con-

juntion with the City's own testing laboratory at the City Hall. The field laboratory contained the necessary equipment for the testing of the aggregates, making of concrete cylinders as well as an insulated curing room in which the cylinders were stored under standard curing conditions. The City's laboratory was equipped with a Riehlé 200,000 pound testing machine and equipment for making tests on cement and other building materials, both physical and chemical.

Cement

All the cement for the job was obtained from the Exshaw Plant of the Canada Cement Company. Sampling was carried out under the supervision of an assistant and tests were made at the City's laboratory. Soon after the beginning of the work we discovered that the strength of the cement at three days was greater than that required by our specifications (A. S. T. M.) for seven days and consequently it was released to the work on the three day test. While the 28 day tests were carried on throughout in every case the actual tests were far above the requirements.

Aggregates

The aggregates for practically the whole of the work except the pipe line were obtained either from the Elbow River deposits or from a near-by pit and were washed and screened in a plant situated at the North end of the Dam. A two-inch graded gravel, A. S. T. M. specification, was used for all the work except a few thin walls or slabs. Cobbles up to 5 inches in diameter were permitted in the Class B concrete of the Dam but were not available in sufficient quantity to warrant their use.

The aggregates were sampled and tested during each shift not only to see that the requirements of the contract were being fulfilled but also to determine the size and grading of the aggregate and the mixture to enable us to properly design the mix for the particular piece of work in which the concrete was being used.

To secure the proper proportions in each batch the aggregates were weighed separately, the cement was put in by the bag and the water was measured in a 35 gallon hot water tank with gauge glass and graduated scale. Due to the fact that the aggregates were used soon after washing the moisture content of the sand was very irregular and the Inspector supervising this part of the work had to make frequent changes in the water to be added. With frequent checks he was soon

able to produce a concrete of the desired consistency by inspection of the concrete within the mixer or in the bucket as it was taken away from the plant. In order to maintain constant proportions once determined the sum of the weights of sand and water was kept constant, the variation in the weight of the stone being very small. That this method was accurate was proved by the exceptionally close agreement of the volumes of concrete as calculated from the Inspector's mixer records with that computed by actual cross sections taken by the engineering staff. Throughout the various structures the concrete was mixed for a minimum period of 2 minutes.

From each sample of concrete two 6-inch by 12-inch cylinders were made and maintained moist at a temperature of 70 degrees Fahrenheit in the curing room until tested, one at 7 days and the other at 28 days. From the numerous tests made the 28 day strength could be fairly closely determined from that shown by the 7 day test by the Formula

$$S_{28} = S_7 + 22\sqrt{S_7}$$

The average 28 day strength obtained throughout the job was given by the Formula

$$S_{28} = \frac{14000}{X^6}$$

where X is the water cement ratio. In the body of the Dam the average 28 day strength of the Class B concrete was 2,000 pounds per square inch obtained by the use of 3.7 bags of cement per cubic yard, the slump being about 2 inches. For the 12 inch facing of the Dam the average 28 day strength was 3,165 pounds, for which $5\frac{1}{2}$ bags per cubic yard were used, the slump being about 3 inches. In the bridge the 28 day strength was 3,590 pounds, using 6.5 bags per cubic yard. For the Water Purification Plant the average strength was 3,680 pounds per square inch in which 6.8 bags of cement per cubic yard were used, the slump being 5 inches to 7 inches.

Aside from the testing of the concrete as incorporated in the work the laboratory was used to carry out many tests to secure information to assist us in the proper carrying out of the work.

At the City laboratory, tests were made of the steel used in the reinforcing rods, structural members and plates, as well as for the cast iron and cast steel used in the various parts of the work.

OPERATION

The plant was put into service on January 9, 1933, and continued under the direction of the Consulting Engineers until the end of that year. The available water power was sufficient to operate the turbine driven pumps throughout and the motor driven pumps were used only for testing purposes. The minimum water level in the reservoir due to normal operation was 3.5 feet below the crest, occurring on March 16, 1933.

During the winter months the quality of the raw water was particularly good, being practically free from B. coli, the total plate count of bacteria 400 per 100 cc. and the turbidity about 0.5 p.p.m. An effluent with a turbidity of 0.1 p.p.m. was produced without a coagulant, but a chlorine dosage of 0.5 pound per million gallons was used largely as a precautionary measure although there appeared to be little reason for it. When the run-off began in April the river above the reservoir became quite turbid and in the course of about a week the turbidity of the water at the Dam increased to such an extent that a clear effluent could not be produced without the use of alum. The turbidity of the raw water reached a maximum of 115 p.p.m. requiring 1.4 grains per gallon of alum for coagulation and at the same time the chlorine dosage was increased to one pound per million gallons.

Although the turbidity reaching the plant after passing through the reservoir was extremely fine a good floc was formed in the mixing tanks and much of it was removed by sedimentation, the filters further reducing it to below 0.5 p.p.m. The color which had risen from 5 p.p.m. in the winter months to about 35 p.p.m. was reduced about 50 percent by the treatment.

Alum was used from April 26 to June 12, at which time the quality of the raw water had improved to such an extent that the filters alone were quite capable of producing a satisfactory effluent without coagulation. It was found that with raw water turbidities below 20 p.p.m. the use of alum was unnecessary. In this regard it is interesting to note the influence of the reservoir on the reduction of the turbidity in the water. Tests were made of the river water entering the Glenmore Reservoir during the months of May and June. This showed that most of the turbidity settles out in the reservoir under normal flow conditions. For example, on June 16 the turbidity of the water entering the reservoir was 410 p.p.m., but the maximum turbidity found in any samples of the raw water entering the plant after this

time was 11 p.p.m. and this was further reduced in the plant to 0.25 p.p.m. without the use of alum. It is therefore evident from the first year's operation of the plant that the great improvement in the quality of the water in the reservoir, both physical and bacteriological, represents a large annual saving in the use of chemicals, which in Calgary are very costly.

The maximum flows of the Elbow River usually come in the early part of June and in preparation for this the flood control valves were tested early in May and partially opened in the latter part of May to reduce the water level in the reservoir to a minimum on June 2, 1933, of 15.35 feet below the crest at which level the storage was reduced 11,400 acre-feet. This was the lowest level at which the water power was sufficient to meet the water demand. Despite the fact that the accumulation of snow in the mountains was above normal the weather conditions were such that the flow in the river never reached anything like flood proportions and the water level was allowed to gradually return to within a few feet of the crest where it was maintained for several weeks to permit of the completion of the pitching on the part of the earth Dam which would be in contact with water.

In the operation of the plant water samples were frequently taken of the raw water, filter effluent and the plant effluent and both physical and bacteriological tests were made in the laboratory. Tests from the month of June when no alum was used showed that the total bacteria of about 3,600 per 100 c.c. in the raw water was reduced 80 percent by the filters alone and this is further reduced 50 percent by chlorination. The filtration process was also found to be capable of eliminating practically all of the B. coli; a small amount of chlorine, about 0.1 p.p.m., however, was required to entirely eliminate this organism.

The quantity of water required for the back-washing of the filters was small, reaching a maximum of 2.1 percent average for May but in June amounting to only 1.1 percent of the water treated.

The City's water consumption is comparatively high, particularly during the hot summer months when a large amount is used for watering lawns and gardens. A certain amount is used in the winter time during periods of severe weather to prevent the freezing of pipes. During such months as April or October when practically no water is used for either of the above purposes the consumption is about 15.5 million gallons per day, which for a population of 85,000 persons

represents a consumption of 182 gallons per capita. On July 27, 1933, the City used 26.75 million gallons, or 314 gallons per capita. The maximum rate of demand in any one hour was 40 million gallons per

TABLE 3

CONTRACT	NAME OF CONTRACTOR	TOTAL AMOUNT
Dam	Bennett & White Construction Company, Limited, Calgary	$1,077,842.53
Watermains	Canada Lock Joint Pipe Company, Limited, Ottawa	571,799.68
Valves	Drummond, McCall & Company, Limited, Toronto	23,167.08
Purification Plant Substructure	Bennett & White Construction Company, Limited, Calgary	295,689.59
Purification Plant Superstructure	J. McDiarmid Company, Limited, Calgary	163,109.38
Purification Plant Equipment	W. J. Westaway Company, Limited, Hamilton	108,718.50
Embankment	Fred Mannix, Calgary	38,055.00
Pumping Equipment	Canadian Allis-Chalmers, Limited, Toronto	76,245.00
Gates, Racks, Conduits, etc.	Dominion Bridge Company, Limited, Calgary	50,734.66
Butterfly Valves	John Inglis Company, Limited, Toronto	28,173.85
Screens	Dominion Bridge Company, Limited, Calgary	13,690.85
Sluice Gates	Dominion Bridge Company, Limited, Calgary	4,458.12
Gate Valves	Drummond, McCall & Company, Limited, Toronto	6,924.36
Pumping Station, Screen House and Chemical Building	Bennett & White Construction Company, Limited, Calgary	190,415.66
Meters	Simplex Valve & Meter Company of Canada, Limited, Toronto	8,076.58
Alum Machines	E. Dean Wilkes Company, Limited, Toronto	3,193.55
Filter Sand	E. J. Couch, Calgary	6,869.53
Filter Gravel	J. E. Jefferies, Calgary	1,932.90

day or 470 gallons per capita. In June the maximum consumption in any one day was 25.37 million gallons. The average consumption over the whole year is about 16.5 million gallons per day, or about 194 gallons per capita.

COSTS

The total cost of the complete works, including land and engineering, was approximately $4,000,000.00, only a small portion of which went outside of the Province of Alberta.

That the construction of this plant during the depression years was of great assistance to the City may be easily understood. For 2½ years many men were employed and laterally mostly on a rotation basis but the maximum was during the summer of 1931 when over 1,000 men were employed each day. With the purchase of materials, many of which were manufactured within the City, many thousands of citizens received benefit.

For the construction of the works and for the supplying of equipment contracts were carried out as shown in table 3.

ENGINEERING

The design of the works, the supervision of construction and the operation for a period of one year after placing the works in service was carried out by the firm of Gore, Nasmith & Storrie, Consulting Engineers, Toronto. A. S. Chapman, the City Engineer, and W. E. Robinson, the Water Works Engineer, gave valuable assistance to the Consulting Engineers during the entire period of design and construction. N. G. McDonald was the Resident Engineer in charge of construction and operation, with R. T. Hollies, Assistant Engineer. The field engineering staff during 1931 when construction was at its height consisted of three engineering parties, two draftsmen and one office man. The inspection staff consisted of two men on sampling and testing of the concrete and materials, one man who sampled the cement at the mill, six men on the mixing and placing of the concrete on the Dam, two on the purification plant and three on the manufacture and laying of the water mains.

The operation of the plant is under the direction of W. E. Robinson, Water Works Engineer, and R. T. Hollies, Plant Superintendent, and J. W. Young, Chemist.

(Presented before the Canadian Section meeting, April 4, 1934.)

THE CONSTRUCTION OF A FIFTY-TWO INCH STEEL TRANSMISSION WATER MAIN IN THE METROPOLITAN AREA OF NORTH EASTERN NEW JERSEY

By P. A. Potter and Joseph J. Domas
(*Hackensack Water Company, Weehawken, N. J.*)

The water supply for fifty-one adjacent communities in Bergen and Hudson Counties, New Jersey, is obtained from a watershed 115 square miles in extent, comprising the upper Hackensack River and its tributaries. The intake, purification plant and main pumping station are at New Milford, in Bergen County. The territory served includes three ridges and two valleys paralleling and west from the Hudson River, south from the New York State line to Hoboken and Jersey City. One third of this area, largely rural, lies north of the pumping station. The remainder, to the south, is suburban to urban in character with maximum density of population and industrial development in the southerly end.

The water, after passing through two impounding reservoirs, a sedimentation basin and the filtration plant is pumped into some 900 miles of transmission and distribution mains.

The transmission system, since its inception in 1882, has gradually been developed by paralleling the original 20-inch cast iron main which extended from New Milford south to Hoboken. The additions, which are two 24-inch, and two 30-inch cast iron mains, following separate routes, are variously interconnected by the secondary grid.

A major reinforcement of the system, with primary interconnections, was undertaken in 1931 after a two year survey and study of pressure and flowage had been made by the Company's Engineers and the resulting comprehensive plan for future development had been approved by Mr. Nicholas S. Hill, Jr., President of the Company.

The first step in this program was the construction of 33,710 feet of 52-inch lock-bar steel main and 3,900 feet of 42-inch cast iron main. This pipe line begins at the 54-inch discharge header of the New Milford pumping station and, following a somewhat angular route south and east, reaches its present terminus in a connection to the easterly 24-inch main of the system at Grand and Sheffield Avenues, in the City of Englewood. In addition, primary interconnec-

tions include 4,200 feet of 30-inch cast iron pipe and varying lengths of other sizes from 24- to 12-inch. Minor side connections to this reinforcing main were limited to a few, none smaller than 8-inch.

Without going into the somewhat complex hydraulics of the whole system it may be said that when this reinforcement was placed in service the hydraulic gradients were raised and materially stabilized, a greater flexibility of control was secured, the effect of any interruption of service in the other mains was minimized and the pumping pressure could be reduced from 125–130 to 110–120 pounds per square inch.

Construction problems of special interest were created by the size of trench necessary in the public streets of four municipalities already occupied by other sub-surface structures, by two railroad crossings and by two river crossings. It was necessary to use a different type of underpass for each crossing. Other features requiring attention in design and execution were: the size and spacing of main valves, blow-off connections, air-vents, expansion couplings, pitometer stations and the operating vaults, of which there are 122.

The work was done under contract after bids had been received on complete plans and specifications prepared by the Company's engineers for lock-bar, hammer-and electric-welded steel pipe and for lock-joint reinforced concrete pipe.

GENERAL FEATURES OF DESIGN

Locomotive fire-box steel, flange quality, A.S.T.M. specifications A. 30–24 was used. All plates are $\frac{1}{2}$-inch thick. Inspection of all materials was made at manufacturer's plant before shipment to the fabricating shop, and again before the finished items were shipped. Cast iron pipe and specials are AWWA Standard Specifications, some flanged, class D, but chiefly bell and spigot, class C.

Single riveted circumferential joints were used in the steel pipe, which was made up in 30 foot sections for all straight clear runs. Fabricated sections were heated to 350° F. and then vertically dipped into "Ovarco" at a temperature of 400° to 450° F.

Bends and specials were designed with long radii. Details of some of the more important bends are: one of 121° with a radius of 67.92 feet made up of 12 segments 5.97 feet long; one of 86°55′ with a radius of 90 feet made up of 17 segments 8.03 feet long. Other bends over 10° were made up of segments 1.75 feet long with radii of 13 to 19 feet. Changes of direction of less than 10° were made by

cutting one or both ends of standard sections at an angle with the axis. No such joint was made with a change of over 8°1′ in direction. Bends requiring thrust anchorage were backed with suitable concrete blocks cast in the trench.

The connection at New Milford is made by a 54-inch flanged cast iron spool attached to a 54-inch by 30-inch cross adjacent to a 54-inch gate-valve previously installed in the original discharge header or manifold outside the pumping station. The 54-inch flanged pipe is dipped under a high pressure main, reduced to 48-inch diameter and is continued to a 48-inch hydraulic gate-valve and then to a 48-inch by 20.25-inch Venturi tube. These appurtenances are set in reinforced concrete vaults. The Venturi meter is connected to recorder, indicator and integrator units mounted on the operating panel board in the filtration plant.

The steel line begins beyond the Venturi meter, and, dipping under the Hackensack River bed, continues east and south to the Overpeck Creek marsh, across which a 42-inch cast iron bell and spigot class C pipe line was constructed to the present terminus.

All main line valves in the steel pipe are 36 inches in size, set in concrete vaults. They are placed before and after each 30-inch side connection and at intermediate intervals of about one mile. Manholes are provided on both sides of such valves and also at intervals of about 1,000 feet to afford convenient access to the interior of the main.

Dresser couplings are inserted in the line on both sides of each valve to provide for expansion and contraction. Pipe was brought home against the ring on the inside of each coupling. Thrust against a valve when closed is taken by struts placed between the sides of the vault and the valve flanges. Observations taken on the couplings during a year show no movement whatever. Main valves and bypasses are operated through standard valve boxes set in the vault roof. A power-driven valve-operating machine mounted on one of the company's special trucks is used to operate all these valves.

Two valves are provided in each side connection, one as close as possible to the steel pipe, the other at the outer end of the connection. This was done to ensure a complete lateral closure at all times, thus obviating the necessity of closing and draining a section of the main.

A blow-off connection was installed at each low point in the line. There are seventeen of these, each consisting of a special casting attached to the side of the main at the bottom, from which extends a 12-inch cast iron pipe with two valves and a vertical tee. The riser

is led from the top of the tee, the bottom of which is plugged to form a convenient sump for unwatering the main.

Air vents housed in special castings are attached where necessary.

Permanent Pitometer stations are located at all strategic points. All of the company's transmission mains are thus equipped for flowage and leak surveys.

All valves over 12 inches, all Dresser couplings, air connections and Pitometer cocks are housed in vaults, nearly all of which are rectangular in section and of dimensions suitable for proper operation and maintenance of these appurtenances. The rectangular vaults are of reinforced 1:2:4 concrete. Others are of brick.

Vault design was standardized in the interest of economy and was based upon considerations of imposed loadings, elimination of live-load impact upon the main, provision for independent settlement of vault and pipe and for expansion and contraction. Loadings were assumed as those imposed by a 20-ton truck according to class "A" highway bridge loadings of the Committee on Reinforced Concrete Highway Bridges and Culverts. Assuming the zone of pressure under concentrated live load to be within slope limits of 1:1 with a maximum unit pressure under the load of 1.5 times the average unit pressure calculated, and designing the roof slabs as simply supported; diagrams were made which simplified the calculations. Resulting slabs vary from 10 to 14 inches in thickness for spans of 10 feet with a reinforcement of $\frac{3}{4}$-inch bars spaced 6 inches on centers. The maximum thickness of vault walls is 14 inches, with $\frac{3}{4}$-inch vertical bars 8 inches on centers. Bottom slabs were designed to support the full load and are identical in section with the roof slabs.

The small vaults are built upon a solid concrete base poured around the lower half of the pipe, a sheet of tar paper preventing adhesion of metal and concrete.

Sumps are provided in all vaults and where ground-water elevations permit, are connected to 4-inch tile blind drains.

Air valve vaults are provided with double covered manholes and the vents have dampers, to prevent feezing temperatures. The dampers are boxes made of sheet copper with shutters to admit or discharge air as required.

EXCAVATION AND SOIL CONDITIONS

The trench, which in part was excavated through private rights of way (later to become streets) and in part through heavily travelled

streets or county roads, was necessarily deep enough to carry the main under existing and projected structures such as gas mains, sewers and house laterals. The cover over the main is therefore from 6 to 10 feet. Thus the operation required that streets be closed to traffic, although bridges were provided for cross streets and public ways.

The route was divided into sections, planned and executed to keep four shovels in continuous operation and to maintain unbroken sequence of excavating, placing, riveting, testing, backfilling and paving in each section. In this way the traffic detours were held within reasonable limits of length and duration. The work in Teaneck Road, through the business section of the town presented many difficulties of traffic and access adjustment.

All excavation was done by two 60 ton Bucyrus diesel-driven shovels, three 30 ton Northwest shovels, all equipped with one-yard buckets, and a Universal crane equipped with a clam-shell for special work. Two of the shovels were rigged as drag-line excavators when working in the open. The excavating machines handled the pipe sections. The larger machines dug soft sandstone that would have required light blasting for a lighter shovel.

About 112,000 cubic yards were excavated, the material ranging fron fine sand or clay to sandstone in some sections.

At the beginning and across the Hackensack River a sheeted trench was carried through fine river sand into tough clay. For the next half mile, through the streets of New Milford, fine dry sand was encountered which with a small amount of bracing caved to an average open top width of 14 feet. Joints were kept exposed by sheeting and sand bags. At the Madison Avenue crossing, a higher water table and heavy traffic required sheeting. Similar sand, but with the water table almost at the surface, was found for $1\frac{3}{4}$ miles south of Madison Avenue. Fortunately this section is chiefly in private right of way so that, except for sheeting at the joints, the trench was allowed to cave.

Then, for a mile, easterly along New Bridge Road the cut was in and out of red sandstone overlaid with red clay and no sheeting was required.

Thence south for one mile, in Teaneck Road, a county highway, there was a mixture of sand and red clay which permitted a vertical trench with no sheeting. The remaining southerly mile in Teaneck Road and a thousand feet east in Cedar Lane gave trouble due to

very irregular sandstone in the bottom of the trench. Rock was excavated 6 inches deeper than the invert grade and an earth cushion placed under the pipe. Rock excavation totalled about 5,400 cubic yards.

The remaining distance, about a mile across the marsh at Overpeck Creek lay through tough clay with a top of fine sand and muck. It was possible to secure a dry trench by using existing roads and dikes as dams and unwatering the entire enclosure.

PIPE-LINE CONSTRUCTION

Five hundred and thirty carloads of pipe were received at convenient sidings. Three 30 foot lengths of the steel pipe constituted a carload. Distribution from the sidings was handled by a local haulage contractor who used motorized cranes to load and unload heavy low-slung trailer-trucks capable of carrying a carload of pipe. Pipe was transported at once from the cars to the route of the line whenever traffic conditions permitted, otherwise it was stored at the sidings until delivered to the shovels.

In New Milford, along the private right of way, it was necessary to carry each length on tandem sleds hauled over the saturated ground by a tractor. At Overpeck Creek, in the marshland, a sled carrying a single length of 42-inch cast iron pipe was hauled out to the trench by a cable strung between two cranes located on farm road dikes on either side of the right of way.

Pipe was lowered onto blocking placed close to the ends. After supporting each section with earth backfill at the middle and when the joint-holes were dug out, this blocking was removed. The main as finally covered thus has no wood near the coating to decay and set up injurious chemical action. Two riveting gangs followed up the pipe placing done by four shovels. Caulking was done by them at the same time. When 30- or 42-inch cast iron pipe was cut, a bead was built up on the spigot end by oxy-acetylene welding.

The pipe coating received special attention. An inspection always was made whenever the pipe was unloaded, stored, moved, riveted and caulked. Any damage to the coating was immediately and thoroughly repaired. After filling and testing, the pipe was drained, all inside abrasions of the coating repaired and a final inspection made. The result was an unbroken coating inside and out from end to end, which justified the constant care required.

Backfilling was done by bulldozers into a puddled trench. The

circumferential joints were not covered until after a pressure test of each section. The bulldozers were heavy and were also used as rough rollers. Where traffic was light and a permanent pavement not required at once a 3-inch layer of cinders was rolled in, under the general contract, and ample time given under traffic and rain for final settlement. Permanent pavement was not included in the general contract and was done when expedient under separate and approved specifications. Bituminous macadam was replaced in Teaneck Road by the County Highway Department immediately after a crushed stone subgrade had been rolled in. This pavement was, in some cases, replaced within three weeks of the trenching.

RAILROAD CROSSINGS

Plans for the two crossings were developed in consultation with engineers of the two railroad companies. Safe and uninterrupted rail traffic was the prime consideration.

At New Bridge Road in Bergenfield the grade crossing is over five tracks of the West Shore Railroad. Considerations of economy and speed precluded the use of an open cut under the tracks with its necessarily elaborate underpinning of the railroad structure. Consequently a circular shield-driven tunnel was designed and constructed as a conduit for the 52-inch steel main. The road-bed was undisturbed.

The completed tunnel is 117.5 feet long, with outside diameter of $114\frac{3}{4}$ inches and inside diameter of 88 inches. The outer shell erected in the tail of the shield is 114 inches inside diameter and made of $\frac{3}{8}$-inch pressed steel plates bolted together. Inside of this is solidly grouted a reinforced concrete pipe cast in 4 foot sections having an inside diameter of 88 inches, within which the main is centered on brick piers. Concrete manholes at each end provide access to plank walks along each side of the main.

Preliminary test holes showed a saturated mixture of fine sand and clay. The area of the crossing was therefore completely unwatered by means of a Moretrench well-point system consisting of 50 well-points on each side of the railroad right of way. Pumping was carried on for a month prior to excavation with the result that the tunnel-face was cut by means of air hand-shovels through compacted sand and clay.

Sheeted shafts were sunk at the manhole sites. The shield, 118 inches inside diameter, was built up of two thicknesses of $\frac{1}{2}$-inch

steel plates riveted together in the west shaft. A welded transverse diaphragm carried the thrust of six 35-ton hand operated jacks which reacted against the tunnel shell. As driving progressed, all voids between the shell and the exterior soil were pressure-grouted at 90-100 pounds air pressure with a 1:2 grout extruded through 2-inch holes spaced 10 feet apart axially. Alternate grout holes served as vents. Progress of the grout was observed by noting the water coming through joints in the shell. All settlement was thus prevented.

Excavated earth was moved from face to shaft over a temporary plank floor by a small scraper and portable air winch. A crane-truck served the shafts. The tunnel was driven in 13 twenty-four hour days.

Upon completion of the shell, a smooth concrete floor was poured in it. The 4 foot concrete pipe sections, weighing approximately 5 tons each were then lowered into the elongated east shaft. Guides were used to enter the lining in the shell, after which a steady pull on the bridle attached about one-third diameter from the bottom hauled each section home, beginning at the west portal. Grouting was done through a 2-inch coupling cast in every fourth section.

The steel pipe sections were lowered into the east shaft and joined from west to east.

The second railroad crossing, under three Erie Railroad tracks at Sheffield Avenue, Englewood, is a rectangular reinforced concrete tunnel, 8 feet by 7 feet inside through which the 42-inch cast iron main is carried on brick piers.

The tunnel has access manholes at each end, is 105 feet long and has walls 12 inches thick with $\frac{5}{8}$-inch bars on 6-inch centers cast between a floor and a roof each 18 inches thick with $\frac{3}{4}$-inch bars on 4-inch centers.

Construction was carried on in a completely sheeted open cut, the railroad tracks being supported on wooden stringers and pilebents. The well-point drainage system was required here also and was equally successful.

The tunnel was divided into three equal longitudinal sections with unbonded, asphaltum painted joints between them. Each section was poured in three operations; floor, walls and roof. The entire floor was completed 30 days after work started. The pipe was placed before the roof was constructed. A non-porous glossy coat of asphaltum water-proofing was applied to the outside of walls and roof.

RIVER CROSSINGS

Substantially similar construction mtehods were followd at the two crossings.

First, temporary timber crane runways were constructed parallel to the site. Steel sheet-piling cofferdams were then driven and the tidal flow carried over in centrally located wooden flumes. Excavation and bracing were then completed.

At the Hackensack River in New Milford the next step was to pour a reinforced concrete base 160 feet long, 12 feet wide and 12 inches thick which covered the area within the sheet piling. Upon this the steel pipe was placed, riveted and caulked. Then a 12-inch reinforced concrete jacket was poured around the pipe and securely anchored to the mat by bars cast into the latter when built. This rigid construction and weight of concrete is more than ample to prevent future lifting or creeping of the main.

At Overpeck Creek the cofferdam was 70 feet long and 10 feet wide. Soil conditions made it necessary to support the 42-inch cast iron main upon pile bents driven six feet on centers. The piles are 12-inch, cresoted and driven to refusal. The caps are 10-inch by 12-inch timbers drift-pinned to the piles. The bents will prevent settlement and lateral movement, while the dip of the pipe from both sides under the creek will resist any tendency of an empty pipe to lift.

Sheet-piling and all temporary construction facilities were removed when testing had been completed.

TESTING

Two tests were specified and made. A pressure test equivalent to a hydrostatic head of 400 feet above mean sea level and a leakage test at an equivalent of 350 feet head. The elevation of the center of the pipe referred to the same datum varies from −6.5 feet under the Hackensack River to +122.5 feet in New Bridge Road.

For testing purposes, dished steel heads were riveted into the circumferential joints at intervals of about 1,400 feet; and after testing were cut up by torch and removed through the pressure manholes. Each section between bulkheads was filled with water from the distribution system when a test was to be made.

The testing outfit consisted of a carefully calibrated tank and a gasoline-engine driven plunger pump permanently mounted on a truck. The pump discharge was connected to the main by flexible

metallic hose attached to rivet drop-holes in the steel pipe or to corporation cocks tapped into the cast iron pipe. Pressures were registered on duplicate six-inch Bourdon test gauges graduated to units of 2 pounds per square inch. Gauges were calibrated by means of a dead-weight testing apparatus after each run.

The outside of the steel pipe was inspected for leaks during the pressure test; no backfilling having been done at joints except where absolutely necessary for traffic. The cast iron pipe was backfilled before testing, and any leaks which developed were dug up and caulked.

Tests were of one hour duration. The specified allowable leakage under the stated head was 100 gallons per inch diameter per mile of pipe per day. Applied to the 6.4 miles of steel pipe this is the equivalent of 1,384 gallons per hour. The actual leakage was 62 percent of this allowance, or 868 gallons per hour, which is equivalent to the amount of water which would pass through a $\frac{3}{8}$-inch orifice at the specified hydrostatic test head.

The Company field office was established at a central location on the job. It was in charge of P. A. Potter, Resident Engineer, assisted by Joseph J. Domas, Designing Engineer, and F. L. Eighmie, Civil Engnieer. The Civil Engineer and party laid out the line, located all structures and checked the line and grade of the pipe as laid. This checking of the pipe was done not only to keep the main to line and grade and to determine settlement, but also that a set of drawings might be made showing the line as actually placed.

Four inspectors, one for each trenching machine, held the pipe to line and grade and kept a record of the progress of the unit as well as the soil conditions including rock excavation. In addition, one inspector specialized on steel plate work, one on concrete structures, one on handling, hauling and painting the steel pipe and one on backfilling.

The Resident Engineer did the testing, gave the entire line one inspection, and looked after public relations. Construction was expedited by the co-operation of the municipal, county and state officials concerned. Public utility companies rendered assistance by furnishing information, replacements and emergency repairs. The project was under the direction of Mr. George F. Wieghardt, Hydraulic Engineer of the Company.

The first pipe was laid July 13, 1931, and the first section of the main from New Milford Pumping Station to Springfield Avenue and

River Road, Teaneck, a distance of three miles was ready for operation in 160 working days. In 205 working days, on March 14, 1932, the main was in service to Queen Anne Road, a total length of 6.6 miles. On March 17, 1932, in 208 working days, the entire main was ready for service.

The General Contract was executed by the T. A. Gillespie Company. Hauling was done by Harper Brothers and pavement restoration by the Ufheil Construction Company.

(Presented before the New York Section meeting, April 20, 1934.)

WATER DISTRIBUTION SYSTEMS

By Charles B. Burdick

(Alvord, Burdick and Howson, Engineers, Chicago, Ill.)

At least half of the water works investment lies in the distribution system. Good service requires that the pipes shall have the ability to serve all the people all the time with adequate pressure at all places. In addition to this, there must be capacity to carry the water at rates required for good fire protection. These things must be done with adequate pressure remaining for the needs of use, and without excessive pressure on the pipes or pumps.

The capacity, and roughly the cost, is based upon the peak rate of flow. The average use per capita has been upon the increase for a long time, but the peak rates have increased much more rapidly especially within the last decade. This has been due to the now almost universal use of the city supply, the more general adoption of water using conveniences in modern homes, better lawns and more flowers, and the more uniform habits of the people. The laboring man now keeps bankers hours. This has resulted in hourly peaks of use nearly double the rates that prevailed fifteen or twenty years ago.

DISTRIBUTION CAPACITY

In small communities, if the requirements of adequate fire protection are met, no difficulty is experienced in supplying the consumers at any consumption rate likely to occur. In cities of from 200,000 to 300,000 population, the supply of the domestic consumers generally governs the size of pipe arteries.

RATES OF USE

It is obvious that unless the distribution system has capacity enough to meet the peak rates of use, the water pressure will suffer. The average use of water is quite variable in different cities depending more particularly upon the extent of industrial use. Average rates of use are readily determined from routine pumping station records. Peak uses, however, are not so readily determined because the extreme peak occurs but seldom. The greatest rates of pumpage gen-

erally occur in exceptionally dry and hot summers. It is often several years between such maximum occurrences.

Table 1 shows average and maximum rates of use in four cities ranging from 43,000 to 300,000 in population. It also shows certain statistics relating to storage, mentioned later. These statistics are presented because they represent cases that have recently been carefully studied. They cover the drought period of 1929 and 30 during which many cities in the Middle West experienced exceptionally high

TABLE 1

Rates of water use and effect of storage in four midwestern cities

	SHEBOYGAN, WIS.	EVANSTON, ILL.	GARY, IND.	LOUISVILLE, KY.
Year considered	1929	1930	1929	1930
Population served	43,000	77,000	95,000	330,000
Percent metered	46	100	100	38
Water pumpage average daily gallons, per capita	140	134	94	125
Maximum 24 hours	218	261	151	212
Maximum hour	420	350	327	352
Variation from average to maximum hours	280	216	233	227
Effect of Storage:				
Elevated storage required to equalize pumpage on day of greatest use, gallons per capita	40	42	42	42
Percent reduction in use of maximum hour on greatest day through use of adequate storage	48	25	56	39
Storage required to secure one-half the percent reduction in last previous line, gallons per capita		12	11	11

pumpage peaks. The pumpage rates in order to be generally applicable are expressed in gallons per capita.

In connection with these statistics, it is interesting to note that the average use of water varies from 94 to 140 gallons per capita. Two of the four cities are completely metered as shown in table 1 and two others are from 30 to 50 percent metered. The maximum hourly use in these four cities exceeds the average use by from 216 to 280 gallons per capita and this variation seems to have very little connection with whether the water is metered, or the average consumption per capita. The excessive demands apparently represent uses and have little to

do with waste. Peak uses may vary in different towns, but the variation is less than might be expected. These data indicate that adding 250 to 300 gallons per capita to the average use will probably cover the peak hour requirements under ordinary conditions.

There are evidently some cities where special conditions bring about much heavier demands for water. The City of Denver, Colorado, is one of these. Here practically all water is sold at flat rates and the semi-arid nature of the locality necessitates very heavy demands for water for lawn and garden irrigation. The average per capita use is approximately 190 gallons. The lower parts of the city are served by a reservoir system and hourly rates of demand cannot be easily determined. However, in the high service districts where there is only a nominal water storage, the hourly peaks of use have reached nearly 900 gallons per capita. The variation in usage thus appears to be about double that which occurs in the ordinary Middle Western city.

PRESSURE

Distribution system pressure must obviously vary with the circumstances under which the water is sold. These variables include the topography of the city, the height of the buildings served in various parts of the town and the distance which the water must be pumped. If 20 pounds is available upon the highest ground or the locality most remote from the pumping station during the hour of maximum demand, the service will probably be satisfactory provided such district is residential. Such remote and high localities are usually residential districts. If this pressure is available under the peak draft conditions, then at ordinary times the pressure will be somewhat greater, the difference representing the pressure necessary to deliver the water from the pumping station to the locality in question.

Twenty pounds available at the hydrant is also regarded as sufficient for fire protection, assuming fire engine pumpers, which are now generally available and assuming, of course, that this pressure is available when the pumping engine is drawing its supply at the maximum rate required by the locality.

PRESSURE LOSSES

Purely theoretical computations of losses in distribution systems are difficult to make because uses vary in different parts of any city, the conditions of the pipes vary and the paths taken by the water to

reach any particular locality are extremely intricate in the ordinary grid iron pipe system. The adequacy of the system can, however, be determined for practical purposes by a test that is not difficult to make. The ordinary procedure in this regard is as follows: A number of fire hydrants are selected at various representative places throughout the city say 30 or 40 hydrants. Locations are spotted on a map; datum elevations of hydrants are determined in advance. A survey of the city is then made by applying a pressure gage successively to each hydrant and recording the pressure, and the time, to the hour and minute. A continuous record of pressure and pumpage is obtained from charts at the pumping station. From 30 to 40 hydrant pressure readings can be obtained by one party in about eight hours. The hydrant readings when tabulated and compared with the station pressure and corrected for elevation indicate the friction loss from the pumping station to each hydrant. For accurate results, allowance must be made for any variations in pressure at the pumping station during the period of the survey, and it is also desirable to make a correction for variations in the rate of pumpage from hour to hour depending upon the hour and minute when the hydrant gage was read and the quantity rate of the station venturi meter. It is sufficiently accurate to assume that friction loss will vary as the square of the pumpage rate in making delivery rate corrections. Having tabulated the hydrant readings and made the necessary corrections, the friction loss can be plated upon a map of the distribution system, and if desired, contours of friction loss may be drawn covering the area of the city, which will show at a glance those localities where increasing the artery sizes would improve the pressures and how much improvement would be obtained.

It is not always feasible to make such a test on a day of maximum consumption, but if the test is made on an ordinary heavy pumpage summer day, it is easy to compute therefrom the losses that would occur at any other rate of pumpage on the assumption, which is approximately correct, that the pressure loss will vary approximately as the square of the water use.

A survey of this kind is most readily applied to a direct pressure system supplied from one source.

CAUSES OF INADEQUACY

Inadequacies are usually caused by growth in the water consumption beyond the capacities of the pipes. This situation is aggravated

from the fact that nearly all iron pipes decrease in carrying capacity as they grow older on account of tuberculation of the interior. According to the well known flow tables of Williams and Hazen, the relation between the age of iron pipe and its capacity to deliver water under average conditions, is as follows:

Age, years	*Delivery, percent*
New pipe	130
5	120
11	110
19	100
28	90
41	80

Admittedly, these figures are no more than rough averages because much depends upon the quality of the water and the nature of the interior coating. The above values are supposed to apply to cast iron pipe as ordinarily coated and laid up to the year 1920 and before.

REMEDIES FOR INADEQUACY

The remedies for excessive pressure losses include, larger arteries at the places where most needed to reduce the distribution losses, the cleaning of the pipe interiors by removal of the tubercles thus restoring the interior to a smooth condition, and the actual reduction of the rates of water delivery by the introduction of elevated storage properly located with reference to the source of supply and the locality of use.

ELEVATED STORAGE

Bearing in mind that pressure losses in the pipe system vary as the square of the water flow in the pipes, that is, when the flow is doubled, the pressure loss is quadrupled, and bearing in mind that peak hour use by the consumers may be from three to four times as much as the average use, it is easy to see how the pressure loss may be cut down through the introduction of elevated storage so located, that at the peak of the consumption of water, the pipes may be fed from both ends. In any pipe that can be thus served, the flow would be cut in half and the friction loss reduced three-quarters. Expressed in another way, the useful capacity of a pipe so benefited would be substantially doubled.

It helps the pipe system very little to have an elevated reservoir located immediately adjacent to the pumping station, although such

a reservoir would be of great aid to the pumping equipment. To be of most advantage to the distribution system, elevated storage should be located on the opposite side of the city from the pumping station. The same beneficial effect can be obtained from several elevated reservoirs skillfully located in the distribution system itself. Topography usually permits of only one large elevated reservoir. There are a few cases where large cities have built elevated tanks located in the distribution system itself in localities where the storage was especially desirable. These elevated tanks are relatively expensive per gallon of water stored, but there are cases in which their use is warranted in the correction of distribution system deficiencies. The City of Milwaukee has installed two elevated tanks and more recently has adopted concrete reservoirs of moderate size located in the distribution system as the most economical procedure for their conditions. These reservoirs are used only on days of heavy pumpage. They are equipped with electric driven pumps. Water is "bled" into the reservoirs at night and is pumped back into the distribution system during hours of excessive demand.

AMOUNT OF STORAGE

It has been previously pointed out that although the average consumption of water is quite variable in different cities, the maximum uses differ less. It is practicable from an examination of the venturi record upon any day of heavy consumption to compute quite accurately the amount of storage required to permit uniform draft throughout the 24 hours. The last three lines of table 1 show the result of this computation in the four cities.

An examination of table 1 shows that in these four cities, storage in the amount of from 40 to 42 gallons per capita would have been effective upon the days of heavy pumpage cited, in permitting a uniform pumpage and would have reduced the peak rate of hourly pumpage by from 25 to 56 percent in the four cities cited. About half of this effect could have been obtained by 11 to 12 gallons per capita in storage.

The excessive consumption rates at Denver have been cited. In that city, about 85 gallons per capita in storage is required for uniform delivery, and such storage would be effective in reducing the peak hour 59 percent in rate of delivery.

The four examples of required storage volume, cited above, were taken from charts covering days when some deficiency in pressure is

known to have occurred at certain places in the distribution systems. It is believed, however, that this deficiency was not great in total water. At Louisville, an opportunity was afforded last summer to check the storage requirements under conditions of very heavy pumpage with no deficiency of service to consumers. The actual rate of consumption could be quite accurately estimated by adding or subtracting from the pumpage, the losses or gains in the reservoir contents from hour to hour.

These figures compare as follows:

	In 1930 before the reservoir was built	*In 1933 with elevated reservoir in use*
Maximum day total in 24 hours, m.g.d.	70	61.4
Peak hour of use, m.g.d.	115	134.0
Storage required to equalize pumpage in 24 hours, gallons per capita	42	38

Thus, while the hourly peak was materially increased, with adequate service, it was a short peak and the storage required to iron out all peaks was not very different from the 1930 estimates.

ADVISABLE CAPACITIES

Years ago when the earliest elevated reservoirs were built, they were deemed to be of great advantage as a reserve source of water for the pumping equipment. Thus, it was customary to build reservoirs sufficiently large to supply the city for several days in case of breakdown at the pumping station. It sometimes happens that an elevated reservoir can be built by damming a natural ravine. In this case, it sometimes happens that a very large reservoir can be built for a comparatively small cost. In these days, however, sanitary requirements necessitate covered reservoirs and it is difficult to justify a reservoir financially that much exceeds a storage sufficient to iron out the inequalities in the hourly demand for water and in addition, has the ability to store such water as may be required for fire protection. In the ordinary city of 100,000 people, a reasonably justifiable capacity in an elevated reservoir is estimated to be approximately as follows:

	m.g.d.
Storage to iron out hourly pumpage peaks at 40 gallons per capita	4
Storage to meet all the requirements of fire protection for an ordinary city of 100,000 population say 12 m.g.d. for a duration of 10 hours—storage required	5
Total	9

To this might properly be added a reasonable allowance for the probable future growth of the city, for such time in the future as might seem to be justified.

CONTROL OF ELEVATED STORAGE

In the smaller cities, elevated reservoirs are generally allowed to float on the system with the reservoir valve wide open. This is practical in the large cities with large reservoirs under ordinary conditions of water demand, or as such reservoirs are usually built with considerable surplus of capacity, many of them are operated at all times with valves wide open.

Where the amount of water in elevated storage is reasonably close to the minimum requirements, it may be necessary in some cases to save the water stored for use during the peak hours of demand. Otherwise, the water might be exhausted before the real emergency occurs. This necessity for controlling the stored water is particularly necessary where elevated tanks are used. In these cases, the item of capital expenditure necessitates rather small volumes of storage.

At Milwaukee, two elevated tanks are controlled through a clock mechanism which opens the tank valve during the filling period, then closes it. It remains closed until a certain hour in the afternoon when the valve is automatically opened allowing the contents of the tank to feed the deficient area surrounding it.

At Louisville, Kentucky, a 30 million gallon elevated reservoir is controlled by an electric valve operated through telephone wires from the pumping station 15 miles away. The mechanism in the pumping station permits and records three functions as follows: A graphical record of the water level in the reservoir, the opening and closing of the valve at will, and the ability to indicate the position of the valve. This ability to promptly control the outlet valve of a large reservoir is regarded as an important point in operation in view of the rather serious consequences ensuing from the breaking of a large pipe in the distribution system, which would make it desirable to close the reservoir valve quickly.

PIPE MATERIALS

As all water works men know, the very great majority of all distribution pipes is, and probably always will be, cast iron. Some experience dating back nearly 300 years and much of it for about 60 years, indicates that it is practically indestructible under ordinary conditions.

Until within the last ten years, the interior coatings used have not been successful in preventing tuberculation with progressive loss in carrying capacity. The use of a cement lining sprayed on gives promise of solving this problem. All experience to date, now covering an increasing use for about 10 years, seems to indicate success.

For large main arteries of the distribution system, say 3 feet and larger, especially where subjected to heavy pressure, the use of steel is growing. It is deemed by many engineers to be better adapted than cast iron to the severe stresses and shocks of the distribution arteries. If it fails, it is less likely to blow out a large piece, and thus a break is less serious. This is an important matter in large cities.

Steel pipes made from sheets are difficult to protect against corrosion and tuberculation, although they can be protected. Within the past decade, a number of large reinforced concrete pressure mains have been laid with splendid success. They appear to have all the advantages of steel for strength, together with the high and sustained carrying capacity of concrete. These pipes are precast in a portable factory and jointed by any means used for iron pipes. Main arteries of this type have been extensively used at Denver, Baltimore, Louisville and elsewhere. Similar pipes have been in use for many years in Europe. For heavy pressures, a thin steel shell about No. 14 gage is embedded in the concrete to insure against leakage.

In common with facilities for water supply, the distribution system must be enlarged progressively with growth in demand. Fortunately, this can be done by adding new arteries and interlacing them with the existing pipes without the abandonment of important pipe. It is advisable, however, to lay down a program of feeders to be built as necessary to meet expected growth, a little in advance of need. An adequate program of capital expenditures, with funds foreseen and set aside, preferably from earnings, is necessary to insure adequate water service in a growing city.

(Presented before the Kentucky-Tennessee Section meeting, March 1, 1934.)

DESIGN OF PIPE LATERALS FOR UNDERDRAINAGE SYSTEMS OF RAPID SAND FILTERS

By J. E. Carson

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The problem of properly designing an underdrain system for a gravity filter has long been troublesome because of the many unknown factors involved. In order to obtain a basis for design, a series of tests was run recently in the hydraulic laboratory of the University of California to determine the combined effect of pressure and velocity in a lateral upon the discharge through a hole therein. Using the curves resulting from this series of tests, a simple solution of the problem was discovered.

EXPERIMENTAL APPARATUS

The test apparatus consisted of a length of 1½-inch wrought iron pipe connected to a stand pipe for maintaining a uniform head. Between control valves placed at the intake and discharge ends of the pipe, holes of various sizes were drilled and a mercury manometer connection was tapped in. Figure 1 shows the details for the experimental set-up. By the proper manipulation of the two valves, various combinations of pressure and velocity in the pipe were obtained. Experiments were conducted upon only one hole at a time, the remaining holes being plugged with cork. A container of known volume was used to catch the discharge of the hole, the time required to fill this container was measured with a stop watch, the discharge of the pipe during the same interval was measured by weight, and the pressure in the pipe was measured with a mercury manometer.

EXPERIMENTAL RESULTS

With the measurements obtained, it was possible to calculate the difference in pressure head between the inside and outside of the pipe, the velocity in the pipe, and the discharge through the hole. The curves shown in figure 2 were plotted to show the discharge of

each size of hole using ordinate values representing the velocity in the pipe and abscissa values representing the difference in pressure head between the inside and outside of the pipe.

An examination of the curves shows them to be straight lines within the limits of the test. An increase in pressure head caused an increase in discharge and an increase in pipe velocity caused a decrease in discharge. The curves for all sizes of holes have nearly the same slope, the greatest variation occurring with the $\frac{1}{4}$-inch hole where the thickness of the pipe probably gave a short tube effect. The experimental data used were limited to velocities between 1 and 10 feet per second in the pipe. A few random points plotted for

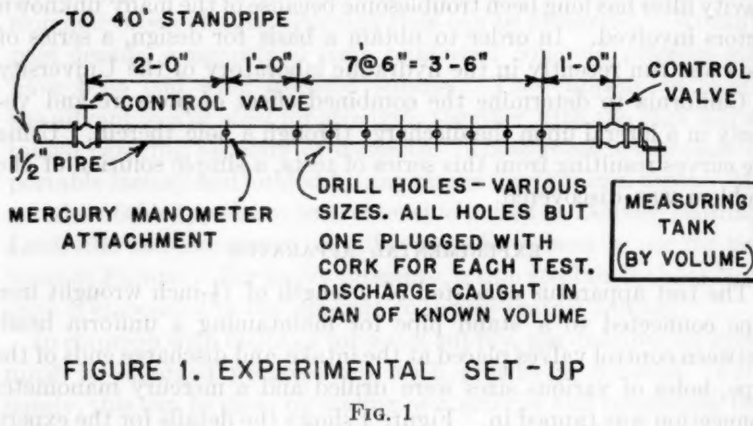

FIGURE 1. EXPERIMENTAL SET-UP

FIG. 1

velocities greater than 10 feet per second indicated that the curves swing sharply to the right, losing their straight line characteristics, with an increase in pipe velocity beyond the stated value. Furthermore, the discharge through the hole diminished rapidly with increased pipe velocities. Therefore, the curves should never be used for pipe velocities greater than 10 feet per second. Velocities above this value are perhaps seldom met with in practice, or at any rate probably should not be.

A few tests were run with the control valve at the outlet end of the pipe completely closed and the results of these random experiments indicate that the curves are accurate if projected downward for velocities of less than 1 foot per second. The minimum velocity would be that necessary to make the discharge at the intake end of the pipe equal to the discharge through the hole or holes.

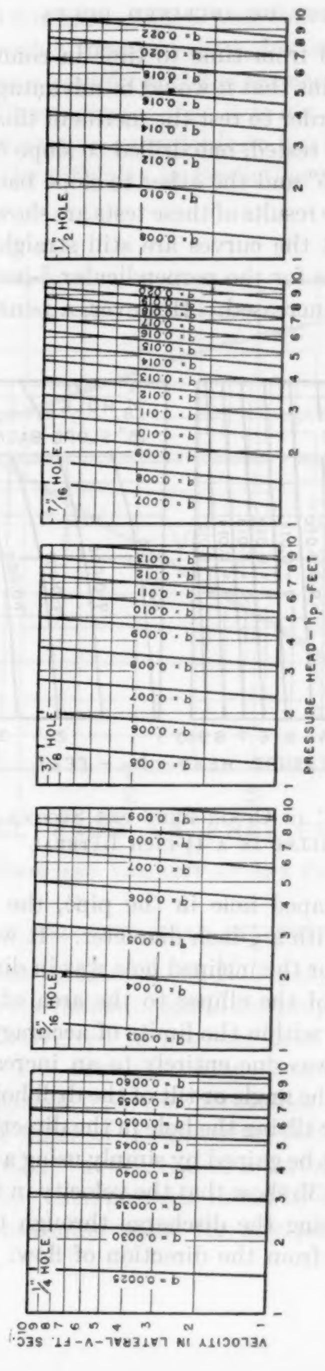

FIG. 2. DISCHARGE "q" IN CUBIC FEET PER SECOND THROUGH HOLES OF VARIOUS SIZE DRILLED IN A 1½-INCH LATERAL

EFFECT OF INCLINED HOLES

It has been suggested from time to time in connection with the design of filter underdrains that it would be advantageous to drill the holes at an angle. In order to test the merits of this idea, two additional ⅜-inch holes were tested, one drilled to slope foreward (i.e., in the direction of flow) 15° and the other to slope backward 15° from the perpendicular. The results of these tests are shown in figure 3a–b.

Figure 3a shows that the curves are still straight lines with the same slope as the curves for the perpendicular ⅜-inch hole, but that the discharge has been increased. However, a ⅜-inch drill tilted 15°

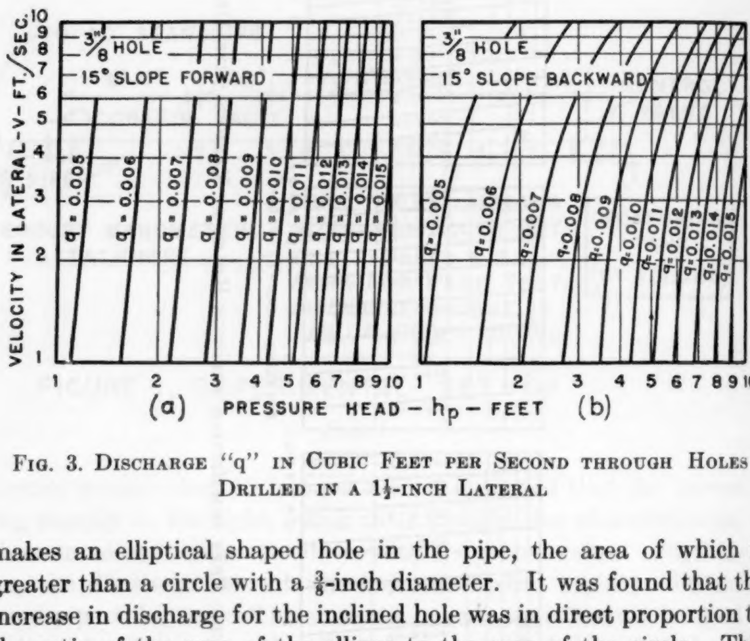

FIG. 3. DISCHARGE "q" IN CUBIC FEET PER SECOND THROUGH HOLES DRILLED IN A 1½-INCH LATERAL

makes an elliptical shaped hole in the pipe, the area of which is greater than a circle with a ⅜-inch diameter. It was found that the increase in discharge for the inclined hole was in direct proportion to the ratio of the area of the ellipse to the area of the circle. This seems to indicate that within the limits of accuracy of the tests, the increase in discharge was due entirely to an increase in area of the hole and none of it to the angle or tilt of the drill hole. Therefore, no advantage is gained by tilting the hole in the direction of the velocity of flow which could not be gained by simply using a larger size of hole.

The curves of figure 3b show that the velocity in the pipe has much more effect in decreasing the discharge through the hole when the hole slopes backward from the direction of flow. For velocities of

1 foot per second it is interesting to note that the discharge is the same as for the hole with a foreward slope, but as the velocity increases, discharge decreases rapidly and the curves lose their straight line characteristic. This indicates that it is not only useless to bore the holes sloping backward in an underdrain but it is a distinct disadvantage because of the difficulty in predicting in advance the behavior of such a hole.

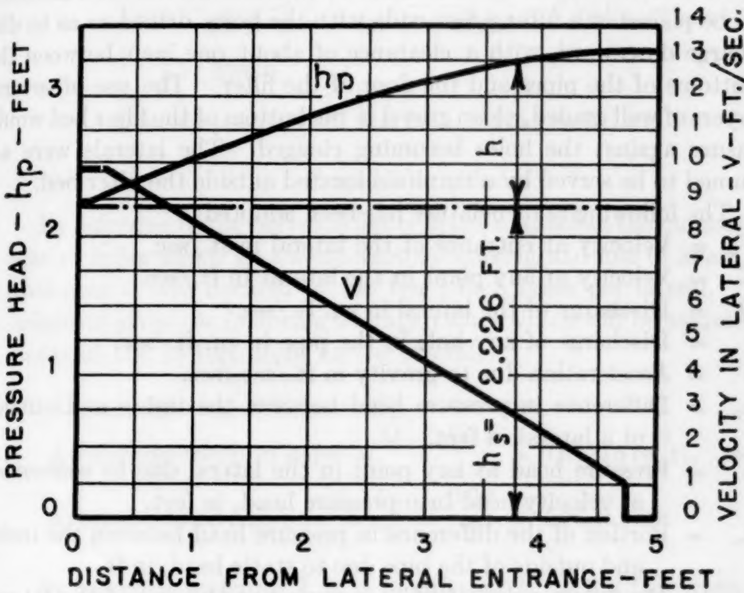

FIG. 4. PRESSURE HEAD AND VELOCITY AT ANY POINT ALONG THE LATERAL

RECOVERY OF VELOCITY HEAD IN PERFORATED LATERALS

In a series of tests on laterals, H. N. Jenks, of the Filtration Division of the City of Sacramento, found the proportion of velocity head which would be converted to pressure head at any point along a perforated pipe. The curves showing these proportions were ommitted in the published report of his studies which appeared in the January 27, 1921 issue of Engineering News-Record. The head recovery factor, "h_{max}," at the end of a perforated pipe, in terms of velocity head at pipe entrance, was found by him to be 0.725. Proportions of this maximum value for any point on the lateral are shown in figure 4.

APPLICATION TO PRACTICAL UNDERDRAIN DESIGN

Although the most important factor for the securing of uniform wash through a filter bed is a sufficient depth of well-graded sand and gravel, it is also important to have uniform discharge under the gravel bed. The purpose of this paper is to show a simple method of design for uniform discharge by varying the spacing of the holes in the laterals.

In the following design, a series of $1\frac{1}{2}$-inch laterals has been assumed to be placed in a filter 5 feet wide with the holes drilled so as to discharge downward, with a clearance of about one inch between the bottoms of the pipes and the floor of the filter. The use of several layers of well graded, clean gravel in the bottom of the filter bed would insure against the holes becoming clogged. The laterals were assumed to be served by a manifold located outside the filter bed.

The following nomenclature has been adopted:

V_0 = Velocity at entrance of the lateral in ft./sec.
V = Velocity at any point in the lateral in ft./sec.
Q = Discharge of the lateral in cu. ft./sec.
q = Discharge of any hole in the pipe in cu. ft./sec.
g = Acceleration due to gravity in ft./sec./sec.
h_p = Difference in pressure head between the inside and outside of a lateral in feet.
h = Pressure head at any point in the lateral due to conversion of velocity head into pressure head, in feet.
h_s = Portion of the difference in pressure head between the inside and outside of the pipe due to static head, in ft.
h_{max} = Maximum value of "h", occurring at the end of the lateral.
A = Area of lateral in sq. ft.
a = Area of hole in sq. ft.
A' = Total area of filter bed served by a lateral, in sq. ft.
a' = Area of filter bed served by any one hole, in sq. ft.
S_L = Spacing of the laterals in feet.
S_H = Spacing of the holes in feet.
L = Length of lateral or width of filter bed in ft.
r = Rate of rise of wash water in vertical ft./min.
n = Number of holes per lateral.

Determination of proper spacing of laterals

For this typical solution, it has been decided to use a rate of rise of wash water of 30 inches per minute, length of laterals, 5 feet, and

diameter of laterals, $1\frac{1}{2}$ inches. It is desired for economical design to have the entrance velocity about 10 feet per second

$$Q = AV_0 = S_L \times L \times \frac{r}{60} = 0.01225 \times 10 = S_L \times 5 \times \frac{2.5}{60}$$

$$S_L = 7.06 \text{ inches (use a 7-inch spacing)}$$

Then:

$$V_0 = \frac{S_L \times L \times r}{60 \times A} = \frac{.583 \times 5 \times 2.5}{60 \times 0.01225} = 9.92 \text{ ft./sec.}$$

$$\frac{V_0^2}{2g} = 1.528 \text{ ft.}$$

$$Q = AV_0 = 0.01225 \times 9.92 = 0.1215 \text{ cu.ft./sec.}$$

Proper size of holes

By referring to the curves in figure 2, one may select the number and size of holes which give a satisfactory spacing and loss of head. In this case it was decided to use ten $\frac{1}{2}$-inch holes per lateral. This selection gives the following average values which will be assumed to occur at the $2\frac{1}{2}$ feet point on the lateral.

$$\text{Average spacing of holes} = S_H = 6 \text{ inches}$$

$$\text{Average discharge of holes} = \frac{Q}{n} = \frac{.1215}{10} = 0.01215 \text{ cu. ft. /sec.}$$

$$\text{Average velocity in pipe} = \frac{V_0}{2} = \frac{9.92}{2} = 4.96 \text{ ft./sec.}$$

By referring to figure 2, it is found that $h_p = 3.0$ feet when the velocity is 4.96 feet per second and the discharge is 0.01215 cubic feet per second.

Pressure head and velocity in a lateral

Using the curve in figure 5, the values of "h" may be computed for the tenth-points along the pipe. These results are shown in table 1.

The value of "h_p" at the $2\frac{1}{2}$ foot point has been found to be 3.0 feet. Therefore, $h_s = 3.0 - .774 = 2.226$ ft. Values for "h_p" at each tenth-point along the pipe may be found by adding the value of "h" to 2.226 feet. These values are shown in table 2 and in figure 5.

The velocity may be assumed to decrease uniformly from the first hole to the last hole. If the correct spacing for the first hole is assumed to be 5 inches and that of the last hole assumed to be 7 inches, the

TABLE 1

Calculated recovery of velocity head in a lateral

Length of lateral = 5.0 feet.
Diameter of lateral = 1.5 inches.
Rate of wash = 30 inches vertical rise per minute.

$$h_{max} = .725 \frac{V_0^2}{2g} = 0.725 \times 1.528 = 1.107 \text{ feet.}$$

DISTANCE FROM PIPE ENTRANCE		RECOVERY FACTOR	$h = 1.107 \times$ R.F.
feet	*inches*		
0	0	0	0
0	6	0.13	0.144
1	0	0.28	0.310
1	6	0.41	0.453
2	0	0.56	0.620
2	6	0.70	0.774
3	0	0.80	0.887
3	6	0.88	0.973
4	0	0.94	1.040
4	6	0.98	1.083
5	0	1.00	1.107

TABLE 2

Calculation of proper spacing of holes in a lateral

Length of lateral = 5.0 feet.
Diameter of lateral = 1.5 inches.
Diameter of holes = ½ inch.

DISTANCE FROM PIPE ENTRANCE		h_p	V	q	SPACING
feet	*inches*				*inches*
0	0	2.226	9.92	0.0100	4.94
0	6	2.370	9.40	0.0105	5.18
1	0	2.536	8.40	0.0108	5.33
1	6	2.660	7.42	0.0112	5.53
2	0	2.846	6.46	0.0118	5.83
2	6	3.000	5.50	0.0122	6.02
3	0	3.113	4.51	0.0124	6.12
3	6	3.199	3.52	0.0127	6.27
4	0	3.266	2.54	0.0131	6.47
4	6	3.309	1.58	0.0135	6.67
5	0	3.333	0.00		

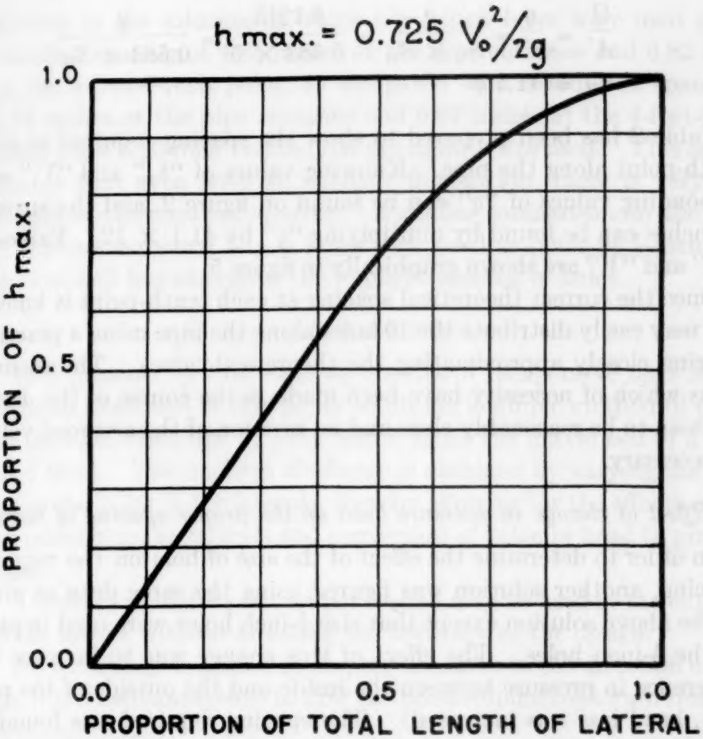

FIG. 5. THE CONVERSION OF VELOCITY HEAD TO PRESSURE HEAD ALONG A PERFORATED PIPE

velocity in the pipe will be 9.92 feet per second for the first 2½ inches and will drop to zero feet per second 3½ inches from the end of the pipe. The velocity at the latter point just before dropping to zero may be computed using the curves of figure 2, as follows:

$$h_p = 3.32 \text{ feet (scaled from figure 5)}$$

Assume $q = 0.0137$ cu. ft. /sec.

$$V = \frac{Q}{A} = \frac{0.0137}{0.01225} = 1.12 \text{ ft./sec.}$$

(This value checks with the value read from the curve.)

Calculation of proper hole spacing

If uniform distribution of wash water is to be obtained, the following relation must be true.

$$\frac{Q}{A'} = \frac{q}{a'} = \frac{q}{S_L \times S_H} = \frac{0.1215}{0.583 \times 5} = \frac{q}{0.583 \times S_H}$$
$$S_H = 41.1\, q$$

Table 2 has been prepared to show the spacing required at each tenth-point along the pipe. Knowing values of "h_p" and "V," corresponding values of "q" can be found on figure 2, and the spacing in inches can be found by multiplying "q" by 41.1 × 12. Values of "h_p" and "V" are shown graphically in figure 5.

Once the correct theoretical spacing at each tenth-point is known, one may easily distribute the 10 holes along the pipe using a practical spacing closely approximating the theoretical values. The assumptions which of necessity have been made in the course of the design are seen to be reasonably close and no revision of the assumed values is necessary.

Effect of change in pressure head on the proper spacing of holes

In order to determine the effect of the size of hole on the required spacing, another solution was figured using the same data as given in the above solution except that size $\frac{3}{8}$-inch holes were used in place of the $\frac{1}{2}$-inch holes. The effect of this change was to increase the difference in pressure between the inside and the outside of the pipe (i.e., head loss was increased). The spacing required was found to vary from 5.53 inches at the pipe entrance to 6.37 inches at the 4 feet-6 inch point. By comparing these values with those in the above solution, it is apparent that increased head loss decreases the magnitude of variation in spacing of holes. This is consistent with results found with previous filter designs in which high pressure heads were used to give good distribution of wash water even though the holes were uniformly spaced.

EFFECT OF FLUCTUATION OF DISCHARGE ON UNIFORMITY OF DISTRIBUTION OF WASH WATER

It might be argued that this method of solution is not desirable because the spacing of the holes is designed for only one rate of rise of wash water and fluctuations in the discharge of the lateral would cause uneven distribution of the wash water. To test the validity of this argument, two additional solutions were computed for extreme variations and the following results were obtained. The substitution of a vertical rise of 18 inches per minute in place of 30 inches per

minute in the solution in which ten $\frac{1}{2}$-inch holes were used gave a required spacing of 4.85 inches at the pipe entrance and 6.82 inches at the 4-feet-6-inch point, as compared with a required spacing of 4.94 inches at the pipe entrance and 6.67 inches at the 4-feet-6-inch point when a 30-inch vertical rise per minute was used. This demonstrates that even with an extreme fluctuation, there is very little change in the required spacing. A similar comparison for the effect of fluctuation when $\frac{3}{8}$-inch holes were used in place of $\frac{1}{2}$-inch holes showed still less change in the required spacing of holes.

CONCLUSIONS

It is possible by first running a series of inexpensive tests similar to those described in this paper to obtain a simple method of design for uniform discharge of wash water under the gravel bed of a rapid sand filter. The uniform discharge is obtained by varying the spacing of the holes in the laterals, thereby allowing for the effects caused by velocity in the laterals and conversion of velocity head to pressure head.

Velocities as high as 10 feet per second may be used at the entrance to the lateral without affecting the accuracy of the design.

No advantage is gained by sloping the holes in the direction of flow and it is disadvantageous to drill the holes sloping backward from the direction of flow.

The velocity of the flow in a lateral reduces the discharge through a hole in the lateral.

By using a series of curves similar to the ones appearing in this paper, it is possible to disregard many of the rules-of-thumb and limiting ratios common to the design of filters in the past.

Using this method of design, larger holes may be used with their advantages of decreased head loss and decreased velocity through the hole. In the past, when the holes were uniformly spaced, it was found necessary to build up large head losses through the use of small holes in order to insure uniform distribution of the wash water.

The performance of a lateral was found not to be affected appreciably by fluctuations in the amount of water passing through it.

DESIGN OF ELEVATED TANKS

By C. R. Young

(Professor of Civil Engineering, University of Toronto, Toronto, Canada)

In these days but little argument is needed in justification of the introduction of elevated tanks or standpipes in water works systems. Added convenience and comfort to the consumer, increased dependability in fire fighting, more economical operation of pumps and extended life for the mains are objectives worth seeking. The question for the municipality is, therefore, not so much whether such a unit shall be installed as it is of the capacity, location and type of unit.

So much of the water stored in a standpipe is relatively inefficient in adding to a high pressure supply that this type of structure has been generally relegated to use at sites where there is a natural elevation, or where the raising of the pressure is secondary to the furnishing of volume. Consequently, elevated tanks have very largely replaced standpipes in modern installations.

While wooden tanks are still used for private purposes and in small public services, for all important installations the two competing materials are now steel and reinforced concrete.

STEEL TANKS

The development of steel tanks has largely turned on the improvement of the bottoms, with the resultant effect on the form and make-up of the supporting structure. While engineers abroad, due to the lack of facilities in the manufacture of dished plates, have been slow to abandon flat bottomed tanks, on this continent the suspended bottom has for many years been considered a standard. Not only does it, for moderate sizes, make possible the saving of material and the reduction of cost, but it makes inspection and painting much easier than is the case with tanks supported by a grillage. In the early stages of steel tank manufacture conical bottoms had a certain vogue, but of late years have been confined almost altogether to railway service, where it is frequently necessary to use muddy water, and where sediment must be removed by a blow-out valve at the bottom.

Hemispherical bottom

For tanks of moderate size, where the question of the variation of head between full and empty tank is not particularly important, the hemispherical bottom continues to be employed. Segmental bottoms possess the disadvantage of the conical bottom, in that an inward horizontal component of force is developed at the point where the bottom joins the shell. Even with a hemispherical bottom, if the amount of water in the tank is small the shape of the bottom tends to be so deformed as to produce compressive circumferential or hoop stresses near the point of junction with the shell. As the head producing such a stress condition is small, however, it is not a governing condition.

One of the objections to the hemispherical bottom is the need for putting in an expansion joint in the riser near its junction with the tank bottom. This requires regular inspection and occasional repacking in order to keep it tight. Moreover, the small size of riser necessitates, in our climate, a frost box to prevent the water in it from freezing.

Ellipsoidal bottom

The desirability of reducing the variation in head as much as possible brought about the introduction, some years ago, of the tank with the ellipsoidal bottom. Such a tank is necessarily one of large diameter and shallow shell. By reason of the flexibility of the bottom it is possible to replace the cast iron riser and its expansion joint by a steel riser riveted to the bottom. As a result of making this riser of large diameter, generally from 3 to 6 feet, the frost box may be eliminated. While freezing occurs to some extent, there is always, even in our climate, a sufficient clear area left to look after the supply of water.

Obviously the rigid attachment of the riser to the tank bottom resolves it into a column to which a certain amount of the water load will be transferred. The correct allocation of load to this so-called "water leg" is a matter of some difficulty.

Radial cone bottom

If the attempt be made to limit the variation of head and still provide a large volume of water, it is found that a tank with a hemispherical bottom would be of very large diameter and practically all

bottom. Consequently, due to the extra work required in the manufacture of dished bottoms, it would be expensive. The use of the ellipsoidal bottom overcomes the difficulty very well for tanks up to about 500,000 gallon capacity. Unfortunately, where the capacity is over about 1,000,000 gallons the compressive hoop stresses in the portion of the ellipse near the junction with the shell, become so great that the depth of the bottom must be increased in order to reduce them. This consequently offsets in part the advantage of the flat ellipsoidal shape.

In this dilemma the designers of steel tanks of very large capacity have been forced back in a sense to the original idea of using a flat bottom. Recognizing the advantages of curvature, however, George T. Horton a few years ago devised the so-called radial cone bottom, which involves the use of a series of conical troughs radiating outward from the riser to the tank shell. Each trough is a portion of the surface of a cone having its apex at the center. For tanks of such capacity as justifies the use of this type, twelve or sixteen troughs are found to be satisfactory. By adjustment of the numbers of the troughs, or by changing their depths, it is possible to control the stresses in the cones within desired limits. At their edges the conical troughs are joined over radial girders which are supported at the riser and by a number of steel columns, usually two. These radial girders act as continuous beams, the columns being so spaced as to bring approximately equal loads to them. It is very necessary, however, to make sure of little or no settlement of the columns lest the assumptions of continuity be upset.

A number of large tanks have been constructed according to this system and have apparently given perfect satisfaction. Among them may be mentioned the following:

	gallons
Sandusky, Ohio	1,000,000
Brooklyn, New York	1,250,000
Indianapolis, Indiana	1,500,000
Evanston, Illinois	1,500,000
Columbus, Ohio	2,000,000
Towson, Maryland	300,000
Tallahassee, Florida	400,000

The Indianapolis and the Evanston tanks each have a diameter of 104 feet with an extreme head variation of only 25 feet.

Roof storage

In the Columbus and Tallahassee tanks advantage has been taken of a portion of the roof for the storage of water. In the former tank the bottom 12 feet are utilized, the supporting roof trusses being on the outside of the roof and giving an unusual and original aspect to the tank. The plates below the high water line are thickened to look after the additional load. In the Tallahassee tank the lower 7 feet of the roof are used for storage.

New column forms

For the Towson tank in which an effort was made to produce a structure in steel, with high artistic merit, the supporting columns were made of hollow section stream-lined in form with a radial depth of 8 feet and a width at the nose somewhat greater than that at the inside edge. They were arc welded throughout, stayed internally by rod trussing, and hermetically sealed in the shop. Supporting the Tallahassee tank are a ring of 33-inch welded cylindrical columns having an unsupported height of 76 feet. Reinforcing diaphragms are welded inside the columns at the top and bottom only.

New bracing ideas

In the Towson tank the lateral support of the hollow stream-lined columns is afforded by horizontal struts only, the usual X-bracing being omitted altogether. The same practice was followed in the tank at the plant of the Ford Motor Company at Cologne, Germany. In the case of the Tallahassee tank there are no X-braces in the exterior vertical faces, but rods were used from the top of a column to the bottom of the second column from it. As these are light they are very inconspicuous as compared with large cylindrical columns. For large tanks, such as those at Evanston and Columbus, the use of stiff riveted bracing between columns is noteworthy. The same practice was followed in the case of the 1,000,000-gallon tank at Kitchener.

Welding of tanks

A growing use of welding is to be noted in the fabrication of steel tanks. In the case of the Evanston, Towson and Tallahassee tanks, all parts were welded except the shell, which was riveted. Recently a 6,000,000-gallon all-welded reservoir tank has been completed for Milwaukee. Apparently this is the most extensive weld job for tank work that has been undertaken up to the present.

There is, however, the need for proceeding carefully in utilizing the welding process in the fabrication and erection of tanks. The failure of an 80,000-barrel arc welded oil storage tank at Tiverton, R.I. has drawn attention to the danger of calking a welded tank when filled. While the stress in the solid plate at the base of a course of this tank was 21,000 pounds per square inch for the test load, with steel having a minimum ultimate strength of 55,000 pounds per square inch, and a factor of safety of only 2.6, nevertheless the failure could not be attributed solely to the high stress. Obviously the calking, or the calk-welding, of a filled tank may be dangerous.

Aesthetics of steel tanks

Some time ago the Chicago Bridge and Iron Works instituted a competition with a view to exploring the possibilities of more artistic tank construction in steel. One of the results of this was the construction of the Towson tank in general conformity with the first prize design. This involves not only the masking of the bottom by carrying the shell down below it as a skirt, but also the carrying over the top of arched ribs in line with, and of the same general form as, the hollow steel columns. The omission of X-bracing has also contributed much to the appearance.

Many steel tanks have, in recent years, been deliberately encased in masonry in order to improve their appearance. One of the most notable of these installations is the group of fourteen steel tanks encased in brick faced concrete at Mount Airy, Cincinnati. The Kennedy Heights unit at Cincinnati utilizes concrete for the masking of a group of five steel standpipes. A particularly attractive housing of a steel tank by a tall reinforced concrete structure with accentuated vertical lines is that at Spokane, Washington. Other attractive treatments are those for the Scarsdale, N.Y., tank, and for the Grover Cleveland Park tank at Buffalo.

REINFORCED CONCRETE TANKS

As a result of many disappointing experiences in connection with the leaking of reinforced concrete tanks as originally constructed, there has been a disposition in recent years to limit such tanks to depths of 20 to 30 feet. Not only will the tensile stresses in the concrete accompanying any reasonable working stress in the hoop steel exceed the ultimate strength of the concrete, without the added effect of shrinkage stress, but the vertical curling effect of a difference in tem-

perature between the inner and outer faces will tend to the production of horizontal cracks. It thus happens that very many tanks of even moderate depth have shown leakage, although, in most cases, they are still serviceable. While no particular harm may result from leakage in warm climates, it is particularly objectionable in a climate such as that of Canada where frost action may aggravate the trouble.

In Europe greater success appears to have been attained with reinforced concrete tanks than has been the case on this continent. Satisfactory performance is credited to the Southall, Hertford and Tilehurst tanks in England, but the depths of all these are moderate. In the case of the Lansdown tank, at Bath, with an interior depth of 11 feet, troublesome leakage developed at horizontal construction joints and less troublesome ones in the floor. Gunite treatment, however, removed the difficulty. As an instance of long service of a reinforced concrete tank there might be cited the dual tanks of the Central of Georgia Railroad, Savannah. This structure, having a total height of 196 feet 8 inches, has two tanks, each about 30 feet deep, one above the other at the top. Although constructed in 1911, and showing a few slight leaks when the tanks were filled, it is still satisfactory.

Cantilever vs. hoop stresses

A complicated stress situation exists near the bottom of a tank wall constructed monolithically with the base. As the diameter of the tank cannot change appreciably under water load at the base, the hoop steel cannot be brought into action to any appreciable extent, and the load is therefore carried by vertical cantilever strips. As the section considered is moved upward the hoop tension increases and the cantilever resistance falls off. At some point, varying from about $\frac{1}{7}$ to $\frac{1}{2}$ the height of the tank wall, the maximum hoop stress arises. The exact determination of this point is a complicated matter, but simplifications of Reissner's method by Carpenter make it possible to locate it easily with only a small error. Naturally, as the diameter of the tank increases relatively to its height, the point of maximum tension will rise since the tank wall becomes straighter and the hoop action less effective. As the thickness of the wall increases relatively to its height, the point of maximum tension also rises since the cantiliver strips become relatively stiffer.

Initial stressing of concrete

The Hewett system of constructing reinforced concrete tanks,

which has been used with apparent success in a considerable number of cases, aims at overcoming the distinctive tendency of concrete in a tank to fail in tension. In accordance with this system the concrete shell is first constructed entirely free of contact with any reinforcing steel, thus taking on a considerable amount of its shrinkage before the steel begins to act. Circumferential compression is introduced in the shell by putting a tension in the horizontal steel which is passed around the shell subsequent to its placing and equipped with the necessary turnbuckles. The appropriate initial stress in the steel depends upon the modular ratio and the steel ratio, but is approximately 80 per cent of the permissible stress. The particular stress adopted for initial tension is such that under water load the stress in the concrete will reach zero at the same time that the stress in the hoop steel reaches its safe value.

To look after differential temperature effects, the vertical steel in the shell, having been previously hooked at the bottom and coated with emulsified asphalt to prevent bonding, is subjected to tension by tightening nuts at the top resting on bearing plates. By so doing such tension may be introduced as will prevent the maximum temperature stress from encroaching upon the safe resistance of the concrete.

Aesthetics

One of the most striking instances of the application of this system is in the Washburn Park standpipe, at Minneapolis. This structure has a diameter of 58 feet 3 inches and a depth of water of 75 feet. Incidentally, a very attractive appearance was produced by the use of broad pilasters carrying moulded figures in concrete with sharply cut lines.

A tank of striking modernistic lines is that at Friedberg-Oberhessen, Germany, for which the surface was given the Contex treatment.

(*Presented before the Canadian Section meeting, April 5, 1934.*)

WATER PURIFICATION—A RETROSPECT

By Wellington Donaldson

(*Of Fuller and McClintock, New York, N. Y.*)

It is with more than usual interest that I address this meeting in Knoxville, because the old Knoxville water works was my first training ground in the field of water supply engineering and water purification, under the kindly tutelage of the late Frank C. Kimball, an able engineer and operator for the Wheeler Interests, then owners of the utility.

The Knoxville plant as it existed about 1903 was interesting in many particulars, and it seems therefore to offer a fitting text or background for tracing some of the significant developments in water purification during the past three decades, developments which have led up to the modern type of plant exemplified by the present works built in 1926. No attempt will be made to deal in any orderly way with the historical facts related to the local supply; this information has already been compiled very interestingly in a paper by the present Engineer-in-charge, Mr. M. B. Whitaker.

The old supply was taken from the Tennessee River through a concrete intake tower adjacent to the old Front Street Station and from there pumped to the purification works on Paine Street Hill. The purification plant with a nominal capacity of 5 m.g.d. comprised a coagulating basin, 18 Warren gravity filters and appliances for dissolving and feeding alum. The filtered water was delivered into twin open distributing reservoirs supplying the City in most part, although a portion of the filtered water was repumped into the high service system which included two standpipes for storage and balancing pressures.

Our chief interest here deals with the filters which had been installed in 1894. During early negotiations over franchise matters the shrewd East Tennessee citizens did not fare so badly in trading with the Boston owners of the water works property, for Knoxville obtained thereby a filtered water supply at a time when filtered supplies were exceptional, instead of common. Slow sand filters, patterned after

European practice, had been in operation at Poughkeepsie since 1872, and subsequently at Hudson, Lawrence and elsewhere, but rapid sand or mechanical filters were just getting well into their stride in 1894. The main business in mechanical filters at that time was the clarification of process water for industries, although there were a number of installations, of pressure type, used for public water supplies, beginning with Somerville, N. J., in 1885. The Hyatt patent involving the use of alum was issued in 1884, and many of the early filters operated as strainers without coagulant, some even with fine gravel instead of sand as a filter medium. Of the representative neighboring cities, it is true that Atlanta had filtered water in 1887; Chattanooga in 1888. Birmingham, destined to become later one of the chief cities of the South, secured filtered water in 1903. The Washington supply was filtered (slow sand) in 1905; Cincinnati in 1907. Louisville, the scene of classic filter experiments by Fuller in 1895–7, did not secure satisfactory filtered water until 1908. New Orleans, after unsuccessful efforts in 1892 by the private water company, obtained a filtered supply in 1909; St. Louis in 1915; Richmond in 1924, while the Nashville supply was not filtered until 1928.

FEATURES OF THE OLD KNOXVILLE FILTER PLANT

The Warren installation at Knoxville was more than an assemblage of stock filter units, it was a patented process developed first in connection with paper mills in New England and later adapted to municipal water supplies. At the time the Knoxville installation was made, doubtless the system represented the best means available for purifying the Tennessee River water which has its muddy moods.

The Warren filters were circular wooden tubs about $10\frac{1}{2}$ feet in diameter with central inlet pipe for the applied water. The tubs had a false bottom of wood members supporting a perforated copper plate which served as a strainer system, and which in turn supported a two-foot layer of coarse sand. A narrow peripheral gutter served to take away the wash water. The filters were equipped with power driven rakes which in washing progressively penetrated the sand bed while revolving, and were withdrawn in the same manner as washing was completed. The filters in operation delivered filtered water into a common control or weir chamber, and in washing a single unit at a time the head in this weir chamber was utilized for back-flushing the filter. The head available for filtration was the difference between the level in the coagulating basin and the level in the weir chamber,

amounting to approximately 2 feet. The sand used in the filters was of very coarse grain according to present concepts, probably in excess of a millimeter in size. Two kinds of sand were experimented with at one time or another, one a natural sand from Lake Sebago, Maine and the other an artificial crushed quartz.

Pre-treatment of the water prior to filtration consisted in dosing with alum and allowing to settle in the coagulating basin with longitudinal baffles. A considerable share of the purification was accomplished in the coagulating basin.

The alum feeding arrangements were simple but effective. There were two wooden solution tanks surmounted by a wooden dissolving tank. The dissolved alum was admitted into the lower solution tanks used in rotation, then made up with water to a standard density according to hydrometer. The alum solution flowed to a constant head box controlled by a float valve and in this box revolved the Warren alum pump. This was a device consisting of six bent tubes which dipped in turn in the float box and in revolving discharged each its quota of solution. Variations in doses were obtained by stoppering one or more of the bent tubes. The pump was driven by a propeller wheel mounted at the inlet of the coagulating basin, and hence acted to proportion the alum according to the rate of pumpage.

In general the Warren installation was quite satisfactory until its capacity became taxed by the increasing demands for water. As might be supposed with such coarse gain filters, fine particles of floc were usually present in the filtrate but these readily deposited in the adjacent distributing reservoirs, so that the public had little complaint to make regarding the quality of water furnished.

IMPROVEMENTS IN FILTER DESIGN

Using the old plant as a basis for comparison, let us now note some of the developments which have taken place in water purification practice. When bacterial efficiency of filters began to be studied and emphasized, it led very naturally to finer grain filter sand, which has now become more or less standardized in present day practice to an effective size of about 0.4 to 0.5 mm. There are however still strong champions of sand both coarser and finer than these limiting values. The use of finer sand resulted in a number of other fundamental changes in filter design. In the first place fine sand could not be held by a simple metal strainer plate punched with holes after the fashion of a nutmeg grater. A number of the early filters embodied brass

strainers having narrow slots. Trouble with clogging of the slots led to the introduction, first at Louisiana, Mo. in 1898, of a gravel layer between the strainer system proper and the sand, to serve as a sand trap. The popular style of filter underdrain for a considerable period consisted of brass strainers with one-sixteenth inch perforations, screwed into the top of manifold pipes and covered with a 10-inch layer of pea size gravel. In some well known installations, as at Erie and Birmingham, the strainers were set in a false bottom plate instead of in the manifold piping.

With the single layer of light gravel more or less shifting of the gravel occurred resulting in inequalities of wash, stoppage of strainers, or loss of sand to the underdrains. The ridge and furrow type of bottom first used at Cincinnati and later installed at Columbus, Grand Rapids, etc., was intended to provide a more permanent underdrain by confining the gravel in definite recesses. The Harrisburg filters designed by Fuertes and installed in 1905 marked the first appearance of perforated lateral pipe underdrains, which are probably the most popular and most widely used type of filter underdrain today. The use of larger perforations or orifices for distribution of wash water led very naturally to heavier gravel layers and to the use of more sizes of graded gravel than had been before necessary. Gravel specifications now call for 15 to 20 inches depth made up of 5 to 7 graded sizes ranging from 2½ inches to 12 mesh. In a number of new plants the larger gravel layers have been replaced by precast concrete blocks as at Omaha, Nebraska and Austin, Texas. The Wheeler bottom made of balls set in pyramidal depressions in the filter floor has been used to a limited extent, for instance, at New Brunswick, and the additions to Little Falls. The same is true of cemented gravel underdrains used experimentally at Sacramento, and the wooden slat bottoms in the second plants at Baltimore and Washington.

Another result of adopting finer sand for filters has been to increase the operating head. Instead of two feet used by the Warren filters, modern filters are customarily designed to utilize 10 to 12 feet of operating head, although the actual loss of head in the sand bed itself probably averages about 8 feet, before the filter is washed.

The use of mechanical rakes and agitators for washing filters is now obsolete for municipal practice although doubtless there are still a few installations being operated. The old filter tubs with mechanical agitators at York, Pennsylvania, were in daily operation until the building of the new plant in 1932 and this equipment is still held in

commission for peak demands. The use of mechanical rakes was not a poor way of cleaning filter beds having a limited supply of wash water, but the use of these rotating devices was naturally limited to circular units of small diameter.

The air wash made its debut in filter practice at Atlantic Highlands, N. J., in 1892 as an alternate to mechanical rakes for loosening the filter bed. Its use with rectangular beds with prevailing low wash rates was necessary but the air wash has declined from its greatest popularity of some 20 years ago. Few question the usefulness of air wash in helping to keep the sand bed clean, but the complications in design and operation account for its disuse in modern design, particularly in view of the tendency to higher wash rates. A notable example of the recent use of the air wash is the San Leandro plant at Berkeley, California.

A notable change in filter plant practice took place when the Little Falls, N. J., and Marietta, Ohio plants went in service in 1902, using reinforced concrete construction and rectangular units. This type of construction marked the passing of circular tub units for municipal water supply because it allowed more compact and pleasing installations, raised the limitations as to size of units and provided the incentive for individual design now reflected so well in the many handsome municipal water works installations, of which the new Knoxville plant is a good example.

The adoption of rectangular units and discarding of mechanical rakes soon led to improvements in washing. Instead of wash rates of less than 12 inches rise per minute, adequate for units equipped with rakes or air wash, it was necessary to increase the wash rates sufficient to float the sand and produce a scouring effect. The change was more or less gradual until a rise of about 18 inches was reached which was generally considered satisfactory. The Cincinnati filters provided for a wash rise of 36 inches, but a rate of 24 inches has been used generally in design practice until the Detroit experiments of Hurlburt and Hering in 1929 showed that much higher wash rates than customary might be desirable in order to keep the beds clean. Some of the new plants are now being designed for a wash rise of 36 inches per minute or higher. With increase in size of filter units and increase in rate of washing, it has been necessary to provide wash water tanks to deliver large volumes of water at high rates without disturbance to the rest of the system, and it has been necessary to increase filter piping, valves and gutters to take care of increased volumes.

The valves on the old tub filters were hand controlled and provided plenty of healthful exercise for the filter operator, for instance Birmingham with its 38 steel tub units, 15 feet diameter. With the increase in the size of units, it became necessary to provide mechanical operation of the filter valves. A few installations, for example Cincinnati had electrically operated valves, but it has now become almost universal practice to equip plants with hydraulically operated valves. To group the controls of these valves on a central table in front of each filter was an easy step which made for convenience and improved the appearance of filter operating galleries, in contrast with the old arrangement of a forest of valve stands.

The old Warren installation was without operating instruments of any kind, or contrivances for regulating the flow through individual filter units.

Between the old filters at Knoxville and the thoroughly modern plant which superseded it in 1926, there was a very wide jump, which illustrates vividly the changes brought about in filter equipment and appurtenances. The effluent pipes of modern filters are equipped with automatic controllers for regulating the delivery of the individual units to a predetermined rate. The flow through each filter is metered and shown by suitable instruments as is the loss of head representing the degree of cleanliness of each bed. The total water handled by the modern plant is accurately metered and automatically recorded instead of being based on the revolutions of a reciprocating pump as was done in the old days.

CHLORINATION

No single improvement in water purification practice has resulted in the saving of so many human lives and so much sickness and disability as the general adoption of chlorination. In this country the practice of disinfecting public water supplies with chlorine began with the Boonton installation of the Jersey City supply in 1908 using hypochlorite of lime. The suitability of this method of treatment, at first questioned, was affirmed by the well known Court Decision of 1909, following which the use of hypochlorite spread rapidly, later to be supplanted by chlorine gas. In 1903 there was no protection against pollution of the supply except that afforded by the coagulation basin, the filters and storage in the open distributing reservoirs. Knoxville started the application of hypochlorite to its water supply in 1912,

and changed over to chlorine gas in 1924. The modern equipment for applying chlorine leaves little to be desired in the way of dependability and accurate regulation.

TREATMENT OF WATER PRIOR TO FILTRATION

The preparation of raw water to make it suitable for filtration has not undergone the striking changes noted in the filters themselves. The old Warren feeding rig was not half bad and aluminum sulfate or filter alum continues to be the principal means of coagulating the suspended impurities in raw water so as to cause them to settle in the basins and the residue to be effectively removed by the filters. Various ingenious devices have been used for regulating the application of alum solution to the raw water but none found so much popularity as the simple system of dissolving known weights of alum in known volumes of water in solution tanks, used alternately, and regulating the dose by means of a constant head box with adjustable orifice valve. This system has been continued in some of the recent large plants, for instance, the new Baldwin filters at Cleveland. Of recent years however with alum being available in powdered form, there has been a strong tendency to use dry feed machines for alum application. The filter installations at Detroit, Buffalo, Richmond and Howard Bend, St. Louis are good examples of this method of handling coagulant. The use of copperas and lime as coagulants in place of alum followed strongly the example set by St. Louis just prior to its World's Fair in 1904. Copperas and lime continue to be used in the New Orleans and Cincinnati plants and has been a popular coagulant with many of the mid-western plants dealing with very muddy water. In the country at large however, particularly in the eastern part of the United States, alum has never had a serious competitor for coagulating water. The experience at Providence however revives interest in the possibilities of iron salts with lime as a coagulant in dealing with problems of manganese removal. It is possible that present ideas about coagulation may be modified in light of work being done with the use of ferric salts, which the English have long employed. The use of chlorinated copperas at Elizabeth City and the experimental work of Black and Bartow in promoting coagulation by the addition of non-hydrous ions suggest possibilities for improving the technique of water coagulation.

MIXING CHAMBERS

Considerable improvement in the preparation of water for filters has been brought about by the recognition that coagulation does not depend upon the simple admixture of the coagulating chemical with the raw water. The advantage of a short period of stirring preliminary to subsidence was recognized in the design of the New Orleans plant (1909) and was followed by many other installations of fixed baffle mixers. The lack of flexibility of these baffled chambers for any considerable range of water flow has led to the increasing popularity of the mechanically-stirred type of mixer or flocculator of which the Sacramento plant was a pioneer, using circular mixers in tandem with paddle stirrers rotating in a horizontal plane. Good examples of mixers of this type are found in the San Leandro plant at Berkeley, Calif., the new Knoxville plant, and the new plant at York, Pa. At Richmond, Va. there has been developed another type mixer or flocculator employing multiple paddle wheels rotating in a vertical plane. The use of this mixer is growing rapidly, as instanced by recent installations at Louisville, Minneapolis, St. Louis and Hamilton, Ont.

LABORATORY CONTROL OF PLANT OPERATION

Thirty years ago there was very little laboratory control of plant operation as we know it today. Sporadic examinations of water supplies were made by a few college professors with a reputation in this class of work. It is a question if these reports had generally any more significance than to advertise the alertness of the local water authorities. The older chemical methods of water analysis, whereby profound deductions were made from the presence of nitrogen compounds to the third decimal place, were giving way to the more direct and certain bacteriological methods. A beginning had been made by the American Public Health Association in 1895 towards standardizing laboratory procedure, then in a state of "rugged individualism." The first report of its Committee, brought out in 1897, dealt only with bacteriological technique and it was not until 1904 that a report covering both chemical and bacteriological methods was brought out. This was the first edition of "Standard Methods," the seventh revision of which appeared in 1933. Bearing now the sponsorship of both the American Public Health Association and the American Water Works Association, this text has a following far beyond the confines of the North American Continent.

The Boston owners of the Knoxville water works being in close touch with the developments along these lines and being fully mindful of their responsibilities to their water consumers, caused to be installed in 1903 a modest though complete bacteriological laboratory, utilizing the annular space existing in the brick substructure of the Paine Street standpipe. The laboratory was fitted out under the supervision of Mr. Robert Spurr Weston, a prominent sanitary expert of Boston, and the speaker assumed under Mr. Weston's supervision, the duties of Water Analyst in addition to his other routine engineering duties. It would be unfair to describe this as a plant control laboratory. It was rather a means of acquiring needed information as to encroachment on the quality of the supply and the capabilities of the filters to meet the situation. Chlorination had not then appeared as an indispensable agent for controlling bacteria and there was really little that could be done by varying the usual plant methods to modify the quality of final effluent.

Today, it is hard to visualize an important filter installation without an adequately equipped laboratory and a full time analyst. The enterprise is run with much closer tolerances than were formerly permissible, and constant vigilance of trained men is required to properly manage the public water supply. Control work is now based definitely on bacteriological procedures, but chemical work plays an important part also. Beside the routine determinations, it seems that the water chemist must concern himself also with such things as iodine which nature sometimes forgot to put in the water in sufficient amount to prevent goiter, and with fluorine that was added in too great amount resulting in mottled teeth.

STANDARDS OF WATER QUALITY

Thirty years ago there was no recognized standard for the quality of drinking water. There are various opinions, of course, but insufficient bacteriological data of water supplies had been correlated with vital statistics to indicate the margins of safety. The analyst of 30 years ago felt rather gratified if dextrose broth showed no fermentation with 1 c.c. inoculations, while now he is apologetic if B. coli is found in 50 c.c.

Certainly one of the most helpful practices in raising the standards of water quality in this country was the promulgation of an official standard for drinking water by the U. S. Treasury Department in 1914. This standard intended to control the quality of water supplied

to interstate carriers later grew to become the yardstick of quality for public water supplies generally. While the Treasury Standard as originally promulgated and as revised under date of 1925 has been criticised as unnecessarily severe in many instances, its far reaching effect in providing safety to water consumers cannot be discounted.

THE BATTLE AGAINST TASTES AND ODORS

In the period around 1903, there was never any serious complaint about palatability of the water supply. Industrial wastes were not a factor and algae propagation in the open distributing reservoirs did not occur to any troublesome extent. This was fortunate because the means then available for controlling tastes and odors were limited practically to the one expedient of aeration. The copper sulfate treatment for the control of algae, now so commonly used, was published by Moore and Kellerman in 1904. The other expedients for overcoming tastes and odors, such as chlorine-ammonia and activated carbon now so extensively used, only appeared within the past decade.

It is worth noting here that Knoxville with its new water plant has followed the trend of the times in discarding open storage of filtered water in favor of covered storage, thus minimizing the opportunity for impairment of the water after filtration by dust, birds, algae, frogs and human carelessness.

There will be another happy landmark in the art of water purification when tastes and odors of public water supplies can be controlled with the certainty that bacterial quality can now be maintained; when Safety and Potability can walk hand in hand with never a jarring note.

TYPHOID

The mortality records of typically water-borne disease constitute the final evidence of safety of a water supply. Taking the registration of the United States area as a whole, the typhoid death rate, during the thirty year period from 1903 to 1932, has been reduced from 34.3 to 3.6 per hundred thousand population. During this same period the typhoid death rate in Knoxville has been lowered from 85.4 to 4.5 per hundred thousand. This is a creditable showing even if Knoxville has not yet reached the honor list of the American Medical Association for cities in which no typhoid death occurred in a single year.

The typhoid statistics just quoted should not be construed to mean that the reduction of typhoid fever is to be attributed solely to improvement in the public water supply. The improvement in medical science and the efforts of various health control agencies have combined with water purification to give this result. At least it is gratifying to note that since adoption of a filtered water supply from the Tennessee River, Knoxville has not suffered devastating epidemics of typhoid fever, such as have visited many other communities.

Taking it all in all Knoxville has done very well in the matter of its public water supply from its beginning to the present. Credit is due the private owners who invested faith, capital and sound engineering experience in the early works which the City was then unable to build or acquire. Credit is due to those conscientious public servants who, since municipal ownership in 1910, have met the needs of this rapidly growing City by improvements to the old works. Credit is especially due also to those responsible for the recent complete modernization of the works and for their efficient management.

(Presented before the Kentucky-Tennessee Section meeting, March 1, 1934.)

ACCOMPLISHMENTS AND OBJECTIVES OF WATER SUPPLY

By Norman J. Howard

(*Director of Water Purification, Toronto, Can.*)

While the earliest provision of public water supplies can be said to date back to the Roman era well over 2000 years ago, it is only during the last century that intensive purification treatment was started which ultimately led to the provision of adequate and potable supplies to millions of consumers, following which the incidence of water-borne disease was decreased from epidemic proportions to the comparatively insignificant figures of the present time.

Few early records are available as to water-borne disease, possibly for two reasons; first, the value of vital statistics was not recognised at the beginning of the nineteenth century, and second, public supplies were obtained from remote sources comparatively free from pollution. As bacteriological research progressed and epidemiological studies showed that many forms of pathogenic microbes were removed in such numbers after filtration, that the probability of water-borne infection was lessened, the practice of filtration became more general, notably in England and abroad.

The major sources of water supply are derived from waters impounded in natural or artifically constructed reservoirs, many of huge dimensions, from lakes, rivers, ground waters and from artesian, deep and shallow wells. Originally most of these were of remarkable organic and bacteriological purity, but as the population became more dense, chiefly in the urban centres, the influence of pollution increased so rapidly that gastro-intestinal disease became common. This condition, coupled with the advances in science and modern-day requirements as to quality of water, resulted in rapid and intensive researches being undertaken.

FILTRATION

The earlier studies were almost entirely confined to the English system of slow sand filtration which, when employed on suitable waters, produced good effluents with greatly reduced bacterial counts. Little change in slow sand filtration has occurred in England,

but in American cities where it is employed, the rate of filtration has been greatly increased up to as high as 10 times that generally employed abroad. The major change in slow sand filtration has been connected with improved mechanical methods of sand cleaning and resanding. Pressure filters, largely developed in America, followed the English filtration method, many types being used, differing somewhat in interior design and methods of cleaning. One filter differing considerably from the rest was the Candy oxidizing filter containing a filter medium consisting of a patented compound known as Oxidium and Polarite. Its use for removing taste and odor and iron was successfully reported upon in England. Following the pressure type, open gravity filters developed rapidly, these again being introduced in the United States and known as the American system of filtration. The early Jewell type was circular in shape but later the filters were made rectangular. The open type of mechanical gravity filters rapidly gained ground, chiefly because of their proven ability to handle the highly turbid waters found in many parts of America. Rapid sand gravity filters can be said to be the beginning of pre-treatment of water whereby sedimentation periods prior to filtration were introduced. In the past twenty-five years mechanical developments and changes in rapid sand filtration have been numerous in all parts of the world. The air-wash system originally developed in America is now little used in that country, but is being increasingly used in Europe. The only other mechanical system of note developed in this period was the drifting sand process as employed at Toronto and elsewhere. This embodies a continuous sand cleaning system in addition to the usual backwashing process. Length of run of filters are, however, much longer than in the standard rapid sand filter. Pre- and double filtration have been employed and found very effective in dealing with water containing heavy bacterial loads or microscopic organisms. The chemical treatment of reservoirs for the prevention and destruction of plankton life has not progressed greatly, but the importance of adequate circulation as a preventive has been well demonstrated.

PRE-CONDITIONING

Arising out of mechanical filtration problems, world-wide studies followed on the size and depth of filter sand, pre-conditioning of raw water, the use of various kinds of chemical reagents for removal of color and for coagulation, artificial methods for mixing chemicals to produce effective flocculation, the mechanical removal of coagulated

sludge, improved methods of backwashing, the elimination of corrosion and greatly improved methods of aëration. The chemical and physical researches in connection with mechanical filtration were largely undertaken in America between the years 1918 and 1930, while other studies of importance were made in England, Germany and France.

The size of filter sand has been the subject of much controversy and largely as a result of experimental work, the original effective size of sand used in rapid sand filters has been increased from approximately 0.3 to 0.6 mm. In the same series of investigations, the relationship of sand expansion to water temperature was featured. The net outcome of these studies has been the production of brighter effluents and increased length of filter runs, with a marked reduction of filter troubles involving mud ball formation, surface cracking and imperfect backwashing. The pre-conditioning of raw water by prolonged storage and chemical treatment has been well established, whilst the use of several new coagulating reagents has simplified the treatment of certain waters previously found difficult to effectively flocculate. In investigating the chemistry of water the researches to determine the optimum point of coagulation known as the iso-electric point, involved a study of hydrogen-ion concentration which threw much light on previously obscure problems relating to coagulation. The outcome of this was to solve the treatment of many difficult waters and prevent corrosive effluents from being discharged into the service mains. In all of these studies three factors have been invariably involved, namely the initial and final pH reaction of the water, the most suitable coagulant and the best method, artificial or otherwise, of producing a floc which would remain stable, attract turbidity and adsorb color. The many types of mixing, including the hydraulic jump, revolving paddles, spiral mix, Aer-o-Mix, baffled basins and several of the Dorr-type, have all materially contributed to the solution of one of the most difficult features of rapid sand filtration. Changes in the baffling of coagulation basins and mechanical methods for removing sludge from sedimentation basins followed improved chemical treatment.

Further progress was made in the removal of iron and manganese from water by means of aëration, coarse contact beds, chemical treatment and filtration. No good reason now exists why a properly treated water should contain either of these minerals in amounts beyond the permissible limits.

SOFTENING

Great strides have been made in water softening, notably in certain parts of the United States and abroad. The economic value, both from a domestic and industrial viewpoint, is enormous. The process now generally employed includes lime and soda, excess lime and recarbonation, and both natural and synthetic zeolites. The subject of corrosion was briefly referred to earlier in this paper and is one of great importance. Chemical treatment with lime is usually employed and has been found to prevent plumbo-solvency and red water in the distribution system. By means of aëration, tastes and odors have been dissipated, while aggressive waters have been made non-corrosive.

DISINFECTION

Having secured clear and non-corrosive water the last problem centred around sterilization without the production of offensive tastes and odors. The introduction of sodium hypochlorite at Lincoln, England, in 1895 marked the first effort to sterilize a public water supply. This was followed by the use of calcium hypochlorite in America in 1908, and the practice of sterilizing water supplies in North America can be said to have become general about 1911. The use of chlorine gas followed in 1917–18 together with the introduction of greatly improved chlorine machines. With the advent of chlorination there followed wide-spread taste in certain localities. These conditions are generally thought to be due to the formation of substitution compounds or the action of chlorine on decomposing organic matter or other substances. Chemical researches rapidly developed means of combating these, chiefly through the medium of super- and de-chlorination, chloramine and, latterly, activated carbon. All have demonstrated their sphere of usefulness but are not regarded as cure-alls. The use of activated carbon for the prevention of taste and odor derived from sources not associated with sterilization treatment has been remarkably successful. The use of ozone and ultraviolet rays has not progressed very rapidly in sterilization treatment, but it is possible that the former may develop in the future.

DECLINE IN TYPHOID FEVER

Following the filtration of water supplies a rapid decline in typhoid fever occurred. In 1890 the American death rate was 80 to 100 per

100,000 of population; in 1906, following an increase in filtration, the rate declined to 32.1 per 100,000, while in 1918 with greatly increased filtration and wide-spread chlorination the rate dropped to 7 per 100,000. The rates in the rural areas are considerably higher than those in the urban centres. At the present time the general United States and Canadian death rate ranges between 4 and 4.2 per 100,000 of population, while the average European rate is somewhat lower.

Briefly then, by artificial processes highly turbid waters have been clarified and made bright and sparkling, color has been removed, water has been softened, corrosion greatly reduced, offensive gases and odors removed, taste forming substances effectively prevented and the danger of water-borne disease greatly lessened. This is chiefly the result of the efforts of the engineering and chemical professions, each playing an integral part and jointly imbued with the spirit of progress and achievement.

FUTURE WORK

The foregoing includes some of the high lights of accomplishment, yet there remain many phases of importance which, due to lack of time have not been touched upon. It is probable that future requirements will follow present-day tendencies to standardise equipment and methods of operation as far as is practicable. New buildings will doubtless be constructed from the architectural viewpoint rather than follow many existing stereotyped designs. The compulsory treatment of sewage and prevention of stream pollution looms up as a policy which in the near future will be enforced, and one which will greatly lessen many existing problems in water purification. Improvement in pre-conditioning of water, in sedimentation basin design, chiefly as regards baffling and mechanical appliances for handling coagulated sediment, will continue to receive increased attention. Radical changes in size and depth of filter sand promise to feature filter studies in the near future. Work carried out with considerable care in Toronto during the past eighteen months, has strongly suggested the use of beds of uniform grades of sand as having definite advantages over hydraulically graded sand at present in general use. It seems probable that water will not be acceptable to the consumer if the color exceeds 14 parts and the turbidity in the treated water is over 0.20 p.p.m. Secondary tastes and odors associated with certain forms of treatment will continue to get less with the control of stream pollution and improved methods of taste

control. Bacteriological standards of purity at present high may be made more severe. Water-borne disease will continue to decrease, but so long as the possibility of accidental pollution or breakdown in sterilization treatment exists, no hope can be held out that the danger of epidemic disease has been removed.

In this paper the writer has purposely omitted the names of well known engineers and chemists who have been closely associated with progress in water supply and treatment. It would, however, be remiss if mention was not made of the important part played by the executives and members of municipal councils, water and public utility commissions and plant operators. By administrative efforts, the former made full provision for an adequate supply and provide the necessary funds to carry on research work and plant maintenance. Much of the progress made has been aided by the efficient coöperation of the plant operators whose efforts are sometimes overlooked.

(*Presented before the Canadian Section meeting, April 6, 1934.*)

MECHANICAL AGITATION AND ALUM FLOC FORMATION

By Carl Leipold

(*Superintendent, Water Filtration Plant, Winnetka, Ill.*)

Our forefathers gave to the water works profession a very useful and economical chemical in alum, which has proved to be indispensable in preparing water for the rapid sand filters. The importance of mixing the water after the application of alum to produce the most effective flocculating action is well recognized. During the past years, various methods of agitation have been used, namely, baffles, hydraulic jump, flocculator and mechanical agitator. For efficient control of the mixing basins, a standard method of laboratory tests with results than can be applied to plant operations, would be of great assistance to the water works engineer. During the past five years, the writer has been making both theoretical and practical studies on the mechanical agitation phase of alum treatment.

Mechanical agitation used for alum floc formation with Lake Michigan water at Winnetka, Ill., was started on June 19, 1928. The original water filtration plant of 3.0 m.g.d. capacity, had one mechanical agitation basin with 10 minutes nominal detention period. The mixer assembly consisted of a three H.P. varying speed, brush shifting single phase motor, a vertical drive speed reducer and a vertical drive paddle shaft with two paddles of 8.40 square feet total surface area or 350 cubic feet basin capacity per square foot of paddle surface area. The velocities at the outer ends of the paddles were variable, 2.4 maximum and 0.95 feet per second minimum, with power consumption of 18 at maximum speed and 12 kilowatt-hours per 24 hours at minimum speed.

The plant extension, put into operation on February 14, 1932, has six mechanical agitation basins with a total combined detention period of 30 minutes at plant capacity of 6.0 m.g.d. The mixer assembly consists of a two H.P. 5-speed 3-phase motor, a horizontal drive speed reducer, one set of bevel gears and a vertical drive paddle shaft. At the time these basins were put into operation, each of the six agitators had one paddle with a total square foot surface area of

6.6 or 409 cubic feet basin capacity per square foot of paddle surface area. Velocities at outer ends of paddle are variable, 2.41 maximum and 0.78 feet per second minimum, with power consumption of 10 kilowatt-hours per 24 hours at maximum speed and also at minimum speed, having one paddle.

LABORATORY EXPERIMENTS

Experiments were made with a Baylis laboratory mixer to determine the possibility of applying the results obtained by the laboratory mixer to the plant mechanical agitators on design of paddle, paddle speed and detention period for alum floc formation.

TABLE 1

Paddles used for laboratory experiments

NUMBER PADDLES USED PER TEST	PADDLE SIZE	NUMBER PADDLES USED PER TEST	PADDLE SIZE
	inches		*inches*
1	$\frac{1}{32}$ x 3	2	$\frac{3}{8}$ x 3
1	$\frac{1}{16}$ x 3	1	$\frac{1}{2}$ x 3
1	$\frac{1}{8}$ x 3	1	$\frac{1}{2}$ x $4\frac{1}{2}$
4	$\frac{1}{8}$ x $4\frac{1}{2}$	1	$\frac{5}{8}$ x $3\frac{9}{16}$
1	$\frac{1}{4}$ x 3	1	$\frac{3}{4}$ x 3
2	$\frac{1}{4}$ x $4\frac{1}{2}$	1	1 x 3

One gallon capacity bottles, round shape, of clear flint glass with wide mouth, metal screw cap type, were used for the containers; and the paddles were made of No. 23 gauge copper sheeting of 12 various sizes (table 1 and figure 1).

The alum solution was prepared by placing 6 grams of ground alum and 200 cc. filtered water into a 250 cc. Erlenmeyer flask (3 percent solution) and whirling the flask by hand until the alum was dissolved.

For velocity or surface water speed tests, a wood float was used (figure 2). After the water in the bottle had been kept in motion for 30 minutes, the wood float was placed at the paddle shaft, above the bottle top, then the wire clip was placed from the opposite side of the paddle shaft, into the holes in the float, and the assembly was dropped into the bottle.

Raw water used for the tests had a pH value of 7.9 and turbidity of 12 and 20.

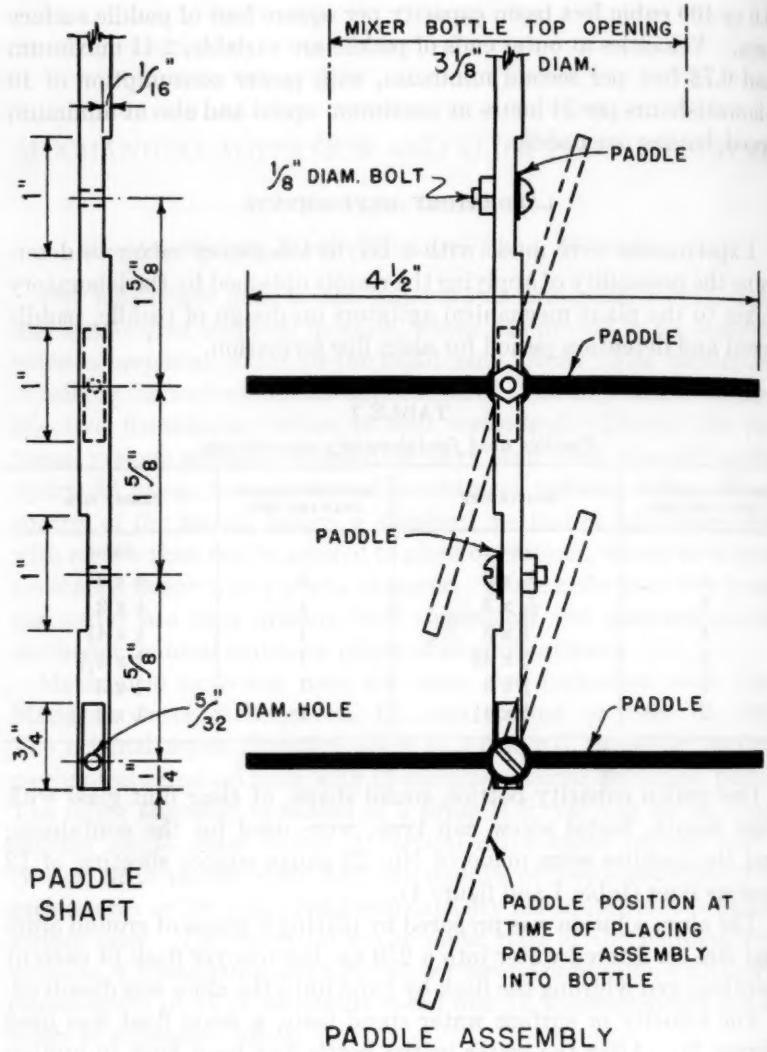

FIG. 1

TEMPERATURES AND PADDLE SPEEDS

Laboratory tests using raw or Lake Michigan water at 5 various temperatures (36, 45, 55, 65, 75°F.) showed that there was no preventive or retarding effect on alum floc formation at low raw water

temperature with 30 minutes mixing, 1.0 grain alum per gallon raw water with pH 7.9 and turbidity 20, paddle size equals one square foot paddle surface area per 8 cubic feet bottle capacity, and paddle speeds at 38 to 140 r.p.m. equals 0.5 to 1.8 feet per second at outer ends of paddle (table 2). The raw water had a temperature of 36°F. at the

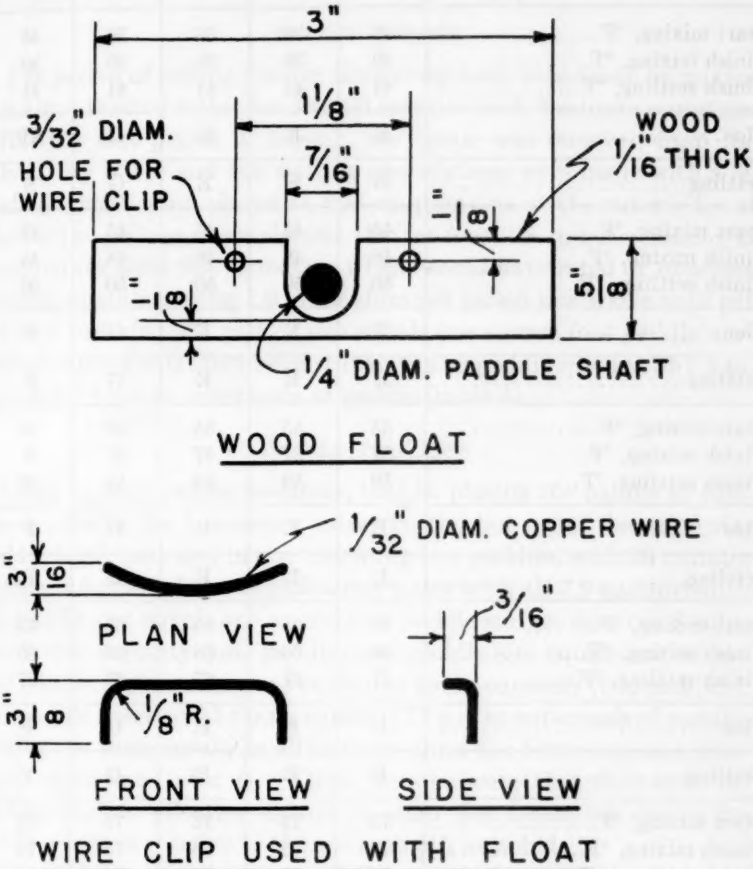

FIG. 2

time of collection for these tests. For the tests on temperatures above 36°F., the bottle was first filled with raw water and then placed in a water bath until the test temperature of the raw water was reached. Floc settling results, after a period of 30 minutes, showed rapid or excellent settling in all tests at paddle speeds below 140 r.p.m. or 1.8 f.p.s. at outer ends of paddle.

TABLE 2

Alum floc test—30 minutes mixing—various raw water temperatures and paddle speeds

	PADDLE SPEED—R.P.M.				
	38	55	70	140	280
Start mixing, °F	36	36	36	36	36
Finish mixing, °F	39	39	39	39	39
Finish settling, °F	41	41	41	41	41
Floc	E	E	E	G	NF
Settling	E	E	E	G	N
Start mixing, °F	45	45	45	45	45
Finish mixing, °F	48	48	48	48	48
Finish settling, °F	50	50	50	50	50
Floc	E	E	E	G	F
Settling	E	E	E	G	F
Start mixing, °F	55	55	55	55	55
Finish mixing, °F	57	57	57	57	57
Finish settling, °F	59	59	59	59	59
Floc	E	E	E	G	F
Settling	E	E	E	G	F
Start mixing, °F	65	65	65	65	65
Finish mixing, °F	66	66	66	66	66
Finish settling, °F	67	67	67	67	67
Floc	E	E	E	G	F
Settling	E	E	E	G	F
Start mixing, °F	75	75	75	75	75
Finish mixing, °F	75	75	75	75	75
Finish settling, °F	75	75	75	75	75
Floc	E	E	E	G	F
Settling	E	E	E	G	F

Alum—1.0 grain.
Raw water: pH, 7.9; turbidity, 20; 3785 cc. = 1 gal.
E = excellent; G = good; F = fair; NF = no floc; N = none.
Settling period: 30 minutes.

Tests also showed that alum floc formation is retarded at paddle speeds above 140 r.p.m. or 1.8 f.p.s. at outer ends of paddle, with 30 minutes mixing, paddle size equals one square foot paddle surface area per 8 cubic feet bottle capacity, and 1.0 grain alum per gallon raw water with pH 7.9 and turbidity 20.

MIXING PERIODS

For period of mixing, twelve laboratory tests were made by mixing the alum treated water for 5 to 60 minutes with 5 minute variation. After the test period of mixing, the bottle was removed from the laboratory mixer and floc settling observations were made with a 60 watt daylight lamp placed at various positions at the outer sides of the bottle. These tests showed that at least 25 minutes mixing is required for alum floc formation which would have rapid or excellent settling qualities, using 1.0 grain alum per gallon raw water with pH 7.9 and turbidity 20, paddle size equals one square foot paddle surface area per 8 cubic feet bottle capacity and paddle speed at 55 r.p.m. equals 0.7 f.p.s. at outer ends of paddle (table 3).

PADDLE POSITIONS

Using various paddle positions, that is, placing the paddle at various depths in the laboratory mixer bottle, but using the same size paddle in all tests and in one test using two paddles, with 30 minutes mixing, 1.0 grain alum per gallon raw water with pH 7.9 and turbidity 12, paddle size equals one square foot paddle surface area per 8 cubic feet bottle capacity (one paddle) and paddle size equals one square foot paddle surface area per 4 cubic feet bottle capacity (two paddles), and paddle speed at 55 r.p.m. equals 0.7 f.p.s. at outer ends of paddle, showed the same results in all tests for alum floc formation and settling of alum floc (table 4). These tests indicate that their is no retarding and no effective change in alum floc formation due to the paddle position relative to the water depth as noted.

The paddle positions were as follows: (A) the paddle bottom was ¼-inch or 3.3 percent of the total water depth, from the bottle bottom; (B) the paddle bottom was 1 inch or 13.3 percent of the total water depth, from the bottle bottom; (C) the paddle bottom was 2 inches or 26.6 percent of the total water depth, from the bottle bottom; (D) the paddle bottom was 3 inches or 40.0 percent of the total water depth, from the bottle bottom; (A and E), using two paddles, the lower paddle bottom was ¼-inch or 3.3 percent of the total water depth, from the

TABLE 3

Alum floc test—various mixing period

MINUTES	TEMPERATURE, °F. Mixing Start	Mixing Finish	Settling Finish	FLOC	SETTLING
5	38	38	42	NF	N
10	38	39	42	NF	N
15	38	40	42	F	F
20	38	40	42	G	G
25	38	40	42	E	E
30	38	41	42	E	E
35	38	42	42	E	E
40	38	42	42	E	E
45	38	42	42	E	E
50	38	42	42	E	E
55	38	42	42	E	E
60	38	42	42	E	E

Raw water: pH, 7.9; turbidity, 20; 3785 cc. = 1 gal.
E = excellent; G = good; F = fair; NF = no floc; N = none.
Alum—1.0 grain. Settling period: 30 minutes.

TABLE 4

Alum floc test—30 minutes mixing—various paddle positions

	PADDLE POSITION A	B	C	D	A and E
Mixing:					
Start, °F.	36	36	36	36	36
Finish, °F.	39	39	39	39	39
Finish settling	39	39	39	39	39
Floc	E	E	E	E	E
Settling	E	E	E	E	E

Raw water: pH, 7.9; turbidity, 12; 3785 cc. = 1 gal.
Settling period: 30 minutes. Alum—1.0 grain.
E = Excellent.

bottle bottom, and the upper paddle bottom was $4\frac{3}{4}$ inches or 63.3 percent of the total water depth, from the bottle bottom.

MIXING PERIODS AND PADDLE SPEEDS WITH VARIOUS PADDLE SIZES

To determine the proper size paddle and paddle speed, and the detention period of mixing required for alum floc formation, various size paddles and various paddle speeds were used for laboratory tests, with 1.0 grain alum per gallon raw water having a pH of 7.9 and tur-

TABLE 5

Alum floc test—various mixing periods, paddle size and speed

(A) PADDLE SPEED 38 R.P.M.	MIXING PERIOD—MINUTES								
	10	20	30	40	50	60	70	80	90
Floc, Paddle "A"—Inches $\frac{1}{32}$	N	N	N	N	N	F	F	G	E
$\frac{1}{16}$	N	N	N	N	F	F	G	E	
$\frac{1}{8}$	N	N	F	G	E				
$\frac{1}{4}$	N	N	G	E					
$\frac{1}{2}$	N	F	E						
$\frac{3}{4}$	N	F	E						
1	N	F	E						
(B) PADDLE SPEED 70 R.P.M.	**MIXING PERIOD—MINUTES**								
	10	20	30	40	50	60	70	80	90
Floc, Paddle "A"—Inches $\frac{1}{32}$	N	N	N	F	G	E			
$\frac{1}{16}$	N	N	F	G	E				
$\frac{1}{8}$	N	F	G	E					
$\frac{1}{4}$	N	F	E						
(C) PADDLE SPEED 140 R.P.M.	**MIXING PERIOD—MINUTES**								
	10	20	30	40	50	60	70	80	90
Floc, Paddle "A"—Inches $\frac{1}{32}$	N	N	F	G	G	E			
$\frac{1}{16}$	N	N	F	G	E				
$\frac{1}{8}$	N	F	G	E					
$\frac{1}{4}$	N	G	E						

E = excellent; G = good; F = fair; N = no floc.
Alum—1.0 grain.
Raw water: pH, 7.9; turbidity, 12; 3785 cc. = 1 gal.

bidity 12 in a bottle of 6-inch diameter and $10\frac{1}{4}$-inch height. The results are shown in table 5, for paddles of 3-inch length.

(A) Using a paddle speed of 38 r.p.m. equals 0.5 f.p.s. at outer ends of paddle.

(B) Increasing the paddle speed 84 percent from 38 to 70 r.p.m. equals 0.9 f.p.s. at outer ends of paddle, showed a reduction of 20

to 37 percent for the detention period of mixing required for alum floc formation with the same size paddles used for the previous tests.

(C) Increasing the paddle speed another 100 percent from 70 to 140 r.p.m. equals 1.8 f.p.s. at outer ends of paddle, showed the same results in all tests for detention period of mixing required for alum floc formation in comparison with the previous tests at paddle speeds of 70 r.p.m., using the same four paddles of size equal to 206, 103, 51 and 25 cubic feet bottle capacity per square foot paddle surface area.

These tests indicate that the detention period of mixing for alum floc formation is determined by the paddle surface area and paddle speed.

VELOCITY

Previous laboratory tests showed that paddle speed is an important factor for alum floc formation. At the present time, the velocity for mechanical agitators is measured by the speed at the outer ends of the paddle.

Laboratory tests were made to determine the effect of various size paddles on the velocity of the body of water being mixed. Using 8 paddles of the same 3-inch length but of various widths, giving a variable paddle size of 6 to 206 cubic foot bottle capacity per square foot paddle surface area, laboratory tests showed the results in figure 3 in surface water speed, with the paddle speed at 70 r.p.m. in all tests.

Placing the paddle at three different positions, that is, placing the paddle at various depths in the laboratory mixer bottle, but using the same size paddle for the 3 tests, showed no change in surface water speed due to the paddle position relative to the water depth as noted.

Using paddles of various sizes, with each paddle having one square foot paddle surface area per 8 cubic foot bottle capacity, the results in surface water speed, with the paddle speed at 70 r.p.m. in all texts, are shown in figure 4.

These tests showed that the paddle surface area in the horizontal direction has a greater effect in producing motion of the body of water than the paddle surface area in the vertical direction. An example is given by the two tests using one $\frac{3}{4}$-inch by 3-inch paddle (paddle length equals 50 percent of the bottle diameter) for No. 1 test, and one, $\frac{1}{2}$-inch by $4\frac{1}{2}$-inch paddle (paddle length equals 75 percent of the bottle diameter) for No. 2 test. The results show the $4\frac{1}{2}$-inch length paddle producing a 25.0 percent greater motion of the body of water than

the 3-inch length paddle, with the paddle surface area being the same in each test.

These tests indicate that the motion of the body of water being mixed is controlled by the number of cubic foot bottle capacity per square foot paddle surface area. The proper design of the paddle,

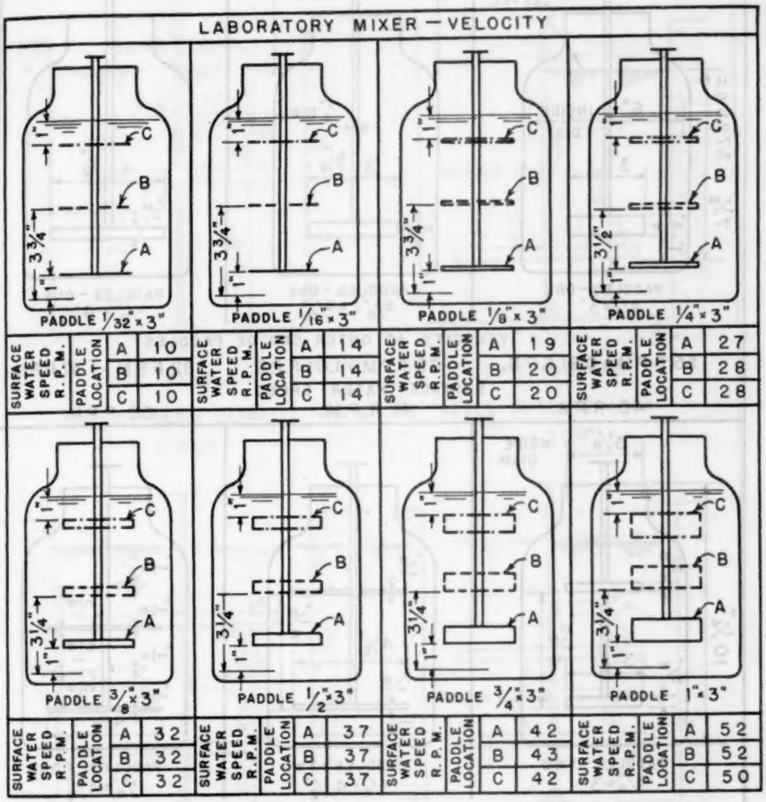

FIG. 3. All tests made with one paddle on shaft and with one gallon of water. Three different paddle locations used. Paddle speed = 70 r.p.m. in all tests. Water agitated for 30 minutes before making surface water speed tests.

therefore, should give due regard to paddle length, percentage of the bottle diameter, and paddle speed.

PLANT OPERATIONS

Experiments were made with the plant mixing basins using one, two and four paddles with various paddle speeds and various detention periods of mixing for alum floc formation (figure 5).

These basins are so arranged that all six basins can be operated in series, or two batteries of three basins each, can be operated in parallel.

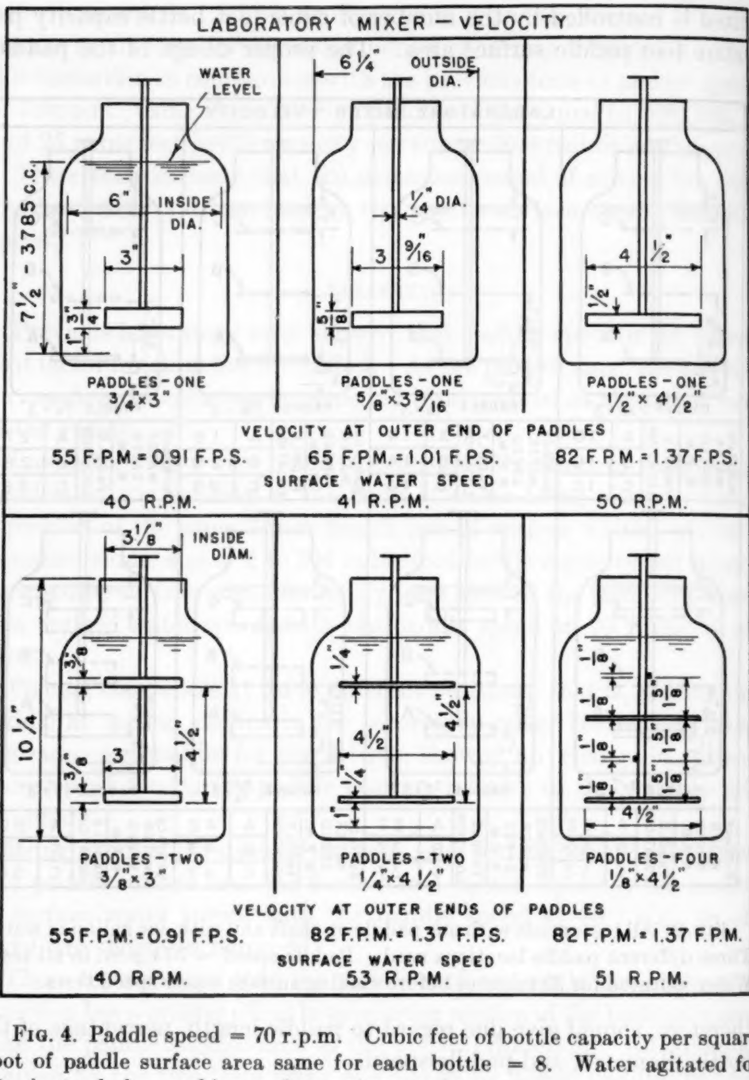

Fig. 4. Paddle speed = 70 r.p.m. Cubic feet of bottle capacity per square foot of paddle surface area same for each bottle = 8. Water agitated for 30 minutes before making surface water speed tests.

An article by the writer on "Mechanical Agitation By Motor Driven Paddles At Winnetka, Illinois," page 103, March, 1933, Water

Works and Sewerage, explains the results of plant experiments using one paddle. For the 82 day test period, the mixing basins were operated in parallel with an average mixing period of 67 minutes and

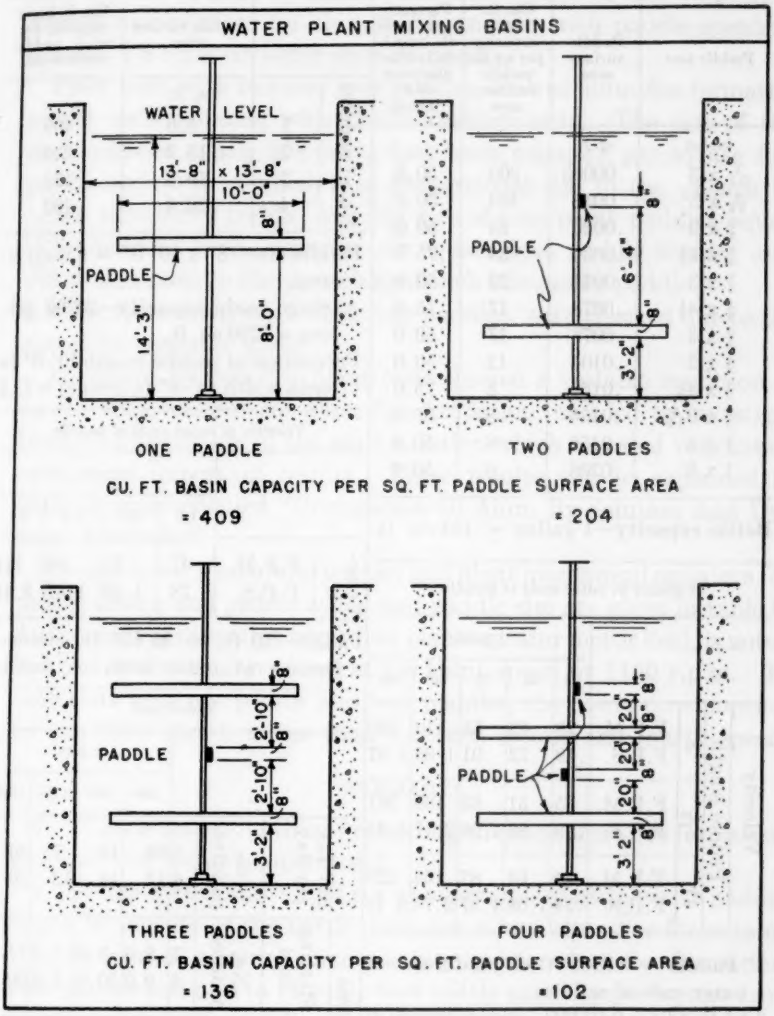

FIG. 5

paddle speeds at the outer ends of the paddle at 1.57 f.p.s. in the first basin, 1.20 f.p.s. in the second basin and 0.78 f.p.s. in the third basin. The plant mixing paddle size equals 409 cubic foot basin

TABLE 6

Comparison of laboratory and plant data

Laboratory mixer

Paddle size	Paddle surface area	Cu. ft. bottle capacity per sq. ft. paddle surface area	Percentage of paddle length to bottle diameter of 6 inches
inches	*sq. ft.*		
$\frac{1}{32}$ x 3	.00065	206	50.0
$\frac{1}{16}$ x 3	.0013	103	50.0
$\frac{1}{8}$ x 3	.0026	51	50.0
$\frac{1}{8}$ x $4\frac{1}{2}$	.0039	34	75.0
$\frac{1}{4}$ x 3	.0052	25	50.0
$\frac{1}{4}$ x $4\frac{1}{2}$	.0078	17	75.0
$\frac{3}{8}$ x 3	.0078	17	50.0
$\frac{1}{2}$ x 3	.0104	12	50.0
$\frac{1}{2}$ x $4\frac{1}{2}$	.0156	8	75.0
$\frac{5}{8}$ x $3\frac{9}{16}$	.0154	8	59.3
$\frac{3}{4}$ x 3	.0156	8	50.0
1 x 3	.0208	6	50.0

Bottle capacity—1 gallon = .133 cu. ft.

Velocity at outer ends of paddle

Velocity	Paddle length		r.p.m. 38	55	70	140	280
Velocity	3″	F.P.M.	29	43	55	110	220
		F.P.S.	.49	.72	.91	1.83	3.67
	$3\frac{9}{16}$″	F.P.M.	35	51	65	130	261
		F.P.S.	.59	.85	1.08	2.17	4.35
	$4\frac{1}{2}$″	F.P.M.	44	64	82	164	329
		F.P.S.	.74	1.08	1.37	2.74	5.49

3″ Paddle = 0.7865 ft. circumference at outer ends of paddle.

$3\frac{9}{16}$″ Paddle = 0.9323 ft. circumference at outer ends of paddle.

$4\frac{1}{2}$″ Paddle = 1.1771 ft. circumference at outer ends of paddle.

Plant mixing basins

		Paddle surface area	Cu. ft. basin capacity per sq. ft. paddle surface area
		sq. ft.	
No. of paddles	1	6.6	409
	2	13.2	204
	3	19.8	136
	4	26.4	102

Paddle size—8″ x 10′-0″ = 6.6 sq. ft. area.

Average basin capacity—20200 gallons = 2700 cu. ft.

Percentage of paddle length 10′-0″ to basin width 13′-8″ (average) = 73.2

Velocity at outer ends of paddle

		r.p.m. 1.5	2.3	3.0	4.6
Velocity	F.P.M.	47	72	94	144
	F.P.S.	.78	1.20	1.57	2.41

Paddle—10 ft. = 31.416 ft. circumference at outer ends of paddle

Motor load

				r.p.m. 580	880	1160	1760
KW hrs. per day		No. paddles	1	10	10	7	10
			4	12	14	12	20
Amperes	Starting load	No. paddles	1	8.6	9.0	7.2	17.6
			4	9.0	10.0	7.5	20.5
	Running load	No. paddles	1	5.0	4.9	2.3	2.8
			4	5.2	5.2	2.6	3.4

capacity per square foot paddle surface area, which is in ratio to the laboratory mixer paddle size of $\frac{1}{64}$-inch by 3-inch. No laboratory tests were made with the $\frac{1}{64}$-inch by 3-inch paddle due to the small size, but the $\frac{1}{32}$-inch by 3-inch paddle tests showed that 60 minutes mixing was required for alum floc formation with paddle speeds of 0.9 and 1.8 f.p.s. at outer ends of paddle.

Plant tests with two and four paddles, showed alum floc formation results corresponding with the laboratory tests. The size of two plant paddles equals 204 cubic foot basin capacity per square foot paddle surface area which is of proportional size to the $\frac{1}{32}$-inch by 3-inch laboratory paddle, and the size of four plant paddles equals 102 cubic foot basin capacity per square foot paddle surface area which is in ratio to the $\frac{1}{16}$-inch by 3-inch laboratory paddle.

Plant tests using more than four paddles, were checked due to the size of the agitator motor.

At certain periods, the plant tests showed a variable condition in mixing requirements for alum floc formation, and led to investigations on preparation of the alum solution which revealed very interesting and important results. These results will be explained in another paper entitled "Preparation Of Alum By Solution And Dry Feed Methods."

Comparisons of laboratory mixer and plant mechanical agitators for paddle design and paddle speed and paddle size are given in table 6. An interesting operation regarding plant agitator motor load, is noted by the low electrical readings at the third speed or 1160 r.p.m. In both tests with one paddle and four paddles, the power consumption for the third speed, is less than the first, second and fourth speed.

SUMMARY

1. There is no preventive or retarding effect on alum floc formation with low raw water temperatures.
2. Paddle speeds above 1.8 f.p.s. at outer ends of paddle with paddle length 50 percent of the bottle diameter retard alum floc formation.
3. At least 25 minutes mixing is required for alum floc formation with paddle size of 5 to 15 cubic foot bottle capacity per square foot paddle surface area, paddle speed at outer ends of paddle at 0.5 f.p.s., and paddle length 50 percent of the bottle diameter.
4. There is no retarding or effective change in alum floc formation due to the paddle position in relation to the water depth.

5. Paddle size is determined by the number of cubic feet bottle capacity per square foot paddle surface area.

6. Detention period of mixing for alum floc formation is determined by the paddle size, and speed or motion of the body of water.

7. Velocity or motion of the body of water, is determined by the paddle size, paddle speed, and paddle length percentage of the bottle diameter.

8. For laboratory mixer control of plant mechanical agitation basins on alum floc formation, it is important that the laboratory paddle size, paddle speed, and paddle length percentage of the bottle diameter, is in ratio with the plant agitator.

9. The method of preparing alum solution for laboratory experiments is very important. Alum solution should be taken from the plant alum solution tanks or the outlet of the dry feed machine dissolving tank.

OPERATION OF THE CARL A. McCLAIN FILTRATION PLANT AT EUGENE, OREGON

By F. Ford Northrop

(*Engineer in Charge of Filtration, Eugene, Ore.*)

The filtration plant at Eugene is named to honor the late Carl A. McClain who for 14 years was General Superintendent-Secretary of the Eugene Water Board. Mr. McClain died suddenly on the day the plant was completed and ready for test, August 5, 1932.

This plant of concrete construction with 12 million gallons daily capacity was built to replace a wooden one that had reached the limit of its capacity and had developed weakness in the wooden settling tanks. It cost $205,000.00 including the value of the land and the equipment moved over from the old plant. It was designed by Stevens and Koon, consulting engineers, of Portland.

The plant is owned by the City of Eugene and operated by the Eugene Water Board, Eugene's commission of water and light. The Water Board also operates two hydroelectric plants, located on the McKenzie river, and a standby steam electric plant in Eugene adjoining the filtration plant.

Eugene got its start in the utility business in 1908, when it was forced to take over the water system as the result of a typhoid fever epidemic. A filtration plant, described as modern, was built soon after and served the city very well for about 23 years. The first hydro-electric plant was built primarily to provide power for pumping and for street lighting, but it was not long until some of the surplus power was being distributed to industrial plants and to domestic consumers.

Although the filtration plant is located on the bank of the Willamette river within the city limits of Eugene, the water supply is obtained from the McKenzie river through a seven mile pipe line. The water supply was changed from the Willamette to the McKenzie river in 1926, as the McKenzie has a considerably lower summer temperature, lower turbidity and lower bacteria content, there being no towns or industrial plants above the pipe line intake.

The seven miles of pipe line cost $306,000.00, not including the intake which cost $28,000.00. The cost of the filtration plant and pumping equipment was only about two thirds of the cost of the pipe line. To get a sufficient gravity supply that would not require filtration or pumping into reservoirs, about 45 miles of pipe line would have been necessary, making the annual carrying charges excessive, besides requiring more storage capacity in the city.

The McKenzie river has a drainage area of 1000 square miles, largely covered with timber. Its upper reaches are among the eternal snows of the Cascade range from which the river gets most of its summer flow. The average low water flow is about 1600, and the maximum recorded flow is 60,000 c.f.s.

In the narrow valley of the river, there is a little cultivated land on which people live throughout the year. The population increases considerably in the summer due to the popularity of the McKenzie valley and Cascade mountains for scenery, fishing, and hunting. The Water Board has no control over the watershed. The upper part is in the Cascade National Forest and the lower is held by private timber interests, except for the summer homes, tourist resorts, and the few farms.

The McKenzie river water is an excellent raw water supply of low turbidity, and usually no color. The water is soft; a recent test showed a calculated calcium carbonate ($CaCO_3$) hardness of 18 parts per million. The characteristics of the raw water at the plant in 1933 are shown in table 1.

PIPE LINE

The intake on the McKenzie river is a concrete structure for screening out the coarser material. The water flows from the river by gravity through gates into three chambers and up through horizontal screens into a collecting flume and thence into the 30-inch welded steel pipe line to Eugene. A trash pump is occasionally operated to remove the sediment from the bottom of the screening chambers.

At the Eugene end the pipe line crosses under the Willamette river and terminates in a control chamber. The pipe line has full flow at all times, the surplus above the plant needs being by-passed to the Willamette river. A meter measures the water to be filtered.

The capacity of the pipe line is about 9 m.g.d. under the 34 feet difference in elevation between low water of the McKenzie and the

water surface in the control chamber. In the future if greater capacity is needed, a pump will be installed at the intake to increase the flow through the pipe line. In the case of an accident to the pipe line, 11 m.g.d., enough water to supply the plant, can be taken from the Willamette river through the condenser circulating pump in the adjoining steam plant. Likewise, the filtration plant can aid the steam plant. The water from the McKenzie can be by-passed through the condensor.

TABLE 1

Raw water characteristics

Temperature, °F.:	
Maximum	60
Minimum	35
Average	48
Turbidity, p.p.m.	
Maximum	400
Minimum	Trace
Bacteria count per cc. Standard Methods nutrient agar 48 hr. 37°C.	
Maximum	480
Minimum	2
Average	33
B. Coli, presumptive test, Standard Methods lactose broth 48 hr. 37°C., percent gas in 10 cc.	
Maximum	46
Minimum	0
Average	8.6
pH	
Maximum (in summer)	7.4
Minimum (during flood)	7.0

TREATMENT

Briefly, the sequence of water treatment in the plant is as follows: The chemicals are added to the water in the control chamber. The water then flows by gravity through a 26-inch pipe 250 feet long to the mixing chambers; thence through a baffle wall into the settling basins. The settled water flume at the far end of the settling basins carries the water into the filter houses where it percolates through the filters into the clear well. From the clear well, electrically driven centrifugal pumps deliver the pure water into the city mains and reservoirs. The plant will filter 16 million gallons per day with clear water conditions as found in the summer.

The plant is rated at 12 m.g.d., which is just about double the present maximum daily pumpage and five times an average winter day's run; so that present operating conditions may be quite different when the plant is at capacity. The plant now serves a population of approximately 20,000.

The chemicals used are chlorine, aluminum sulphate and hydrated lime. Provision is also made for the use of ammonia if desired. Ammonia with chlorine was of value in disinfecting the plant at commencement of operation. Pre-chlorination is practiced, using a vacuum type solution feed chlorinator. Chlorine is admitted to the pipe leading from the control chamber and is fed at such a rate that a residual of 0.1 p.p.m. is maintained at the outlet of the mixing chambers, after a retention period of 30 to 40 minutes. Pre-chlorination is used to reduce the number of bacteria before filtering. It is feared that chlorination after filtration would give an objectionable taste to customers residing near the plant, the nearest customer being only about 600 feet from the pumps. The dosage of chlorine varies from 2.0 to 3.0 pounds per million gallons, the average for 1933 being 2.29. The maximum dosage occurs during times of turbid water and when alum is used.

It is believed that Eugene was the first city in the Pacific Northwest to use chlorine in water purification, the chlorine being in the form of chloride of lime at the beginning. The Oregon State Health Officer at that time very strongly opposed it on the grounds that it would be injurious to health. Since that time not a single case of typhoid has been traced or believed to have been due to drinking the city water. The chloride of lime treatment was recommended and later supervised by O. F. Stafford who is a professor in the chemistry department of the University of Oregon.

To maintain a clear sparkling water in the clear well, alum was required only 55 days in 1933. When the Willamette river was the raw water supply, alum was necessary throughout the year. On only three days was the raw water turbidity over 200 p.p.m. The alum dosage is being determined from Hazen's curve for fine turbidity as given in Stein's "Water Purification Plants and their Operation," 1915, and reproduced in Hopkin's new book, "Water Purification Control." The maximum dosage of alum was 220 pounds per million gallons, when the turbidity was at its maximum of 400 p.p.m. "Jar tests" have shown that the best floc is obtained when the pH of the mixed water is from 6.4 to 6.6. The optimum pH is taken at 6.6.

Just enough lime is added during the periods of greatest turbidity and greatest alum feed, to bring the pH up to this optimum. Lime was used for 19 days in 1933.

The alum and lime are fed into the control chamber outlet from dry feed machines. The machines are set at a higher elevation than the control chamber and streams of water carry the chemicals to their point of application just below the weir between the two halves of the control chamber, where the turbulence of the falling water provides good mixing.

In the mixing chambers, motor driven agitators are installed. They will probably be needed when the plant nears capacity, but at present the low rate of flow in the 250 feet of pipe from the control chamber to the mixing chambers seems to give ample time for thorough mixing to provide a good floc. It takes approximately 3 minutes for the water to pass through the pipe. By the time the water with its slow and circulating movement has come to the top of the chambers, coagulation is complete and settlement commenced.

From the top of the mixing chambers, the treated water passes along a perforated baffle wall and thence through it into the settling basins. The perforations are 4-inch diameter holes separated on 24-inch centers. The baffle effectively spreads the water across the ends of the settling basins as shown by the depth of sediment. Sedimentation begins in the mixing chambers where the sediment accumulation has the greatest depth and then lessens gradually as the water passes across the basins.

At the present rate of use the settling time is from 12 to 16 hours, reducing the turbidity to less than 10 p.p.m. When the plant reaches rated capacity the settling time will be 4 hours. The capacity of the settling basins is 2,000,000 gallons. They are washed once a year, at the end of the winter rainy season.

The settled water flume takes water from the top of the settling basins and conveys it to the filters. Provision is made for 8 filters, 6 being equipped at present. Each has 30 inches of sand and 18 inches of gravel of graduated sizes. The under drains are perforated cast iron pipes. About one-half of the sand is from the old, discarded plant where some had been in use for 20 years. Repeated washing has increased its uniformity and roundness till each sand particle resembles a tiny pearl.

Of the six filters now used, four are in operation, the other two standing idle till the next washing, when they replace the two just

washed. Just the four are used at present in order to keep the rate of filtration as nearly equal to the rate of pumping as possible; even so the rate of flow controllers are set at their minimum.

The filters are rated at 1.5 m.g.d. or 125 million gallons per acre per day, but will pass 2 million gallons per day of clear water.

Two filters are washed each day. This provides for washing after 48 hours service and 24 hours settling before being put back in service. Thus each filter is washed every 72 hours. This washing schedule has given a consistently low bacteria count in the filter effluent and keeps the sand in a clean condition. It has been found that a regular schedule of washing gives better results than to regulate the washing schedule by loss of head. The loss of head in 48 hours is very small.

The wash water pump of 4150 g.p.m. capacity takes water from the clear well. It is started by push button stations on the operating tables. All valves are hydraulically operated. The wash water rises through the filters at the rate of 24 inches per minute and gives a sand expansion of 50 percent. This rate of wash carries a little sand over into the wash troughs, so the rate is kept a trifle lower than this, a little over 40 percent expansion. The crest of the wash troughs is 24 inches above the settled sand. Approximately 2½ percent of the filtered water is used in washing filters.

A submerged light in the clear well indicates the efficiency of filtration in removing solid matter. Tests in 1933 showed an average daily count of 6 bacteria per c.c. in the clear well, and daily B-coli presumptive tests were negative in every instance. There has been a noticeable improvement in the bacteria content since the 72 hour washing routine was established last August.

An interesting feature of these filters is a window built into the side of one of them. There may be observed the sand and gravel layers, and the sand expansion during washing.

Three electrically driven centrifugal pumps of 3, 4, and 6 m.g.d. capacity are installed to take the water from the clear well and deliver it directly to the city mains. The mains in turn are connected to three reservoirs in different parts of the city. The reservoirs are concrete covered and have a total storage capacity of 6 million gallons, and are at an elevation of 185 feet above the clear well, except the Skinner Butte reservoir which is 4 feet higher than the other two. The two lower reservoirs have automatic controlling altitude valves which close the inlet pipes into the reservoirs when full, and the higher one has a long distance altitude recorder connected to the plant.

The pumps are operated during the daytime and evening at maximum discharge, the pump or pumps being selected to give the maximum hours operation per day. The pumps are shut down when the long distance recorder indicates the Skinner Butte reservoir is full. The pressure in the mains in the business district is approximately 75 pounds per square inch.

TABLE 2

Cost of operation of filtration and pumping plant (in dollars), 1933

	TOTAL COST	PERCENT OF TOTAL COST	COST PER MILLION GALLONS
Alum, 15,813 lb. @ 1.6¢	253.00	1.47	0.26
Lime 1,256 lb. @ 1.2¢	15.07	0.09	0.02
Chlorine 2,484 lbs. @ 10.8¢	268.27	1.56	0.27
Power for high pressure pumps—875,000 kwh. (0.86¢ kwh.)	8,516.21	49.63	8.57
Power for wash water, mixer, and sump pumps, and chemical feed machines	534.60	3.11	0.54
Electricity for lighting, chlorinator heating and office heating	951.75	5.55	0.96
Self-consumed water (hydraulic valves, irrigation, washing settling basins, etc.—729,900 cu. ft. @ 7.26¢ per 100 cu. ft.) (Regular commercial rate)	529.60	3.09	0.53
Miscellaneous supplies and expense (phone, tools, records, janitor supplies, etc.)	225.54	1.32	0.23
Maintenance of building and equipment	198.06	1.15	0.20
Salaries of 4 operators, exclusive of meter repairing	4,561.36	26.59	4.59
Supervision, including bacteriologist	1,106.30	6.44	1.11
Total	17,159.76	100.00	17.28

Daily records are made of the principal plant operations and conditions. These include which pumps were operated and length of time, kilowatt hours consumed in pumping, quantity of water treated and pumped, chemicals consumed, time of filter washing, quantity of water used in washing, condition of raw and treated water, water temperature, and maximum and minimum outside air temperature.

By making the operation as much as possible a matter of routine, it is expected that a high quality of water will continue to be delivered, and at the same time lower the cost of supervision. Maintenance of a certain residual in the mixing chambers, regular washing of the filters, and use of curves in determining the amount of coagulants are a step in that direction.

The water delivered to the mains is clear, sparkling, cool, and with no objectionable taste. During 1933, not a single complaint was received as to taste, except in the locality of an open reservoir due to the development of algae in the reservoir. Last December a concrete cover for this reservoir was completed so no more trouble is expected in that vicinity.

No "red water" or rust complaints in cold water have been received, but studies are being made to determine if it would be advisable and economical to raise the alkalinity of the filtered water. "Dead ends" in the distribution mains are flushed regularly once a month.

The total amount of water pumped in 1933 was 994,000,000 gallons. The cost of operation is shown in table 2. The charges for electricity are figured the same as for any private customer. The approximate cost per capita for the year was 90 cents. The cost per 100 cubic feet pumped was 1.3 cents.

Four operators are employed to operate the filters and the pumps. Mr. E. H. Hotaling is chief operator. These men also test and repair water meters to which part of their time is charged. The plant is designed so that, if desired, it may be operated in conjunction with the standby electric plant.

Dr. E. D. Furrer, local pathologist, supervises the bacteriological examination of the water. Daily tests are made of the raw water, filter effluent, and clear well. Weekly test samples are taken from the distribution system.

The writer supervises the filtration processes and records. The general operation and maintenance is under the supervision of W. J. Moore, superintendent of the water department, and J. W. McArthur, general superintendent-secretary of the Eugene Water Board.

(*Presented before the Pacific Northwest Section meeting, May 11, 1934.*)

UTILIZATION OF ALKALI WATERS

By W. M. Cobleigh
(*Dean of Engineering, Montana State College, Bozeman, Mont.*)

Municipal engineering practice and procedures are influenced by the occurrence of alkali salts in the soils and ground waters of certain regions of the state of Montana. These salts occur in soil that has been formed under arid or semi-arid climatic conditions, and in mineral deposits laid down in a saline sea. Naturally ground waters may have a high concentration of alkali salts in solution in localities where soil alkali is abundant. The composition and properties of alkali salts will be discussed in another portion of this paper.

Alkali salts in water and soil have effects in a wide range of engineering practice. The engineer who is considering a highly mineralized ground water for public supply should have the chemical analysis interpreted from several viewpoints: the physiological effect of alkali or mineral salts in solution; the hardness of the water, caused by the calcium and magnesium content; and the undesirable iron and manganese compounds in solution. If the water is to be used for steam raising or other industrial applications, the chemical analysis will furnish a basis for classifying it from the standpoint of scale formation and foaming ingredients, corrosive properties, and the possibility of caustic embrittlement of steam boiler plate. In cases where alkali water and soil come in contact with concrete engineering structures, precautions should be taken to hinder the penetration of alkali solutions into the interior of the concrete in order to prevent the destructive action of alkali salts on the chemical compounds in set cement. Finally the chemical analysis of a mineralized water furnishes a basis for predicting the effect on the soil of the mineral salts in solution, and on plant life in cases where such a water is to be used for irrigating trees, shrubbery, lawns, and gardens.

Composition and analysis of alkali waters

In the laboratory investigation of a typical alkali water to determine its general properties and adaptations a limited examination is usually sufficient. It is customary to determine the amounts of the

following basic radicals: Iron and aluminum oxides (Fe_2O_3, Al_2O_3), Calcium (Ca) Magnesium (Mg) sodium and potassium (Na+K), calculated together, and the following acid radicals: silica (SiO_2) sulphate (SO_4) Nitrate (NO_3) chloride (Cl) carbonate (CO_3) and bicarbonate (HCO_3). A radical is defined as, "a part of a molecule consisting of an atom or a group of atoms which act as a unit in chemical reactions." Basic and acid radicals exist in alkali waters as ions, which in turn are electrically charged. Basic and acid radicals combine chemically to form salts. In reporting the results of a water analysis it is customary to calculate the hypothetical combinations of basic and acid radicals, in order to show the specific mineral or alkali salts that would be formed by evaporating the water to dryness. When the analytical report is made in this manner the following compounds and salts will be found in solution in the various types of alkali water occurring in this state:

Silica	SiO_2
Iron oxide and alumina	$Fe_2O_3 \cdot Al_2O_3$
Calcium bicarbonate	$CaH_2(CO_3)_2$
Calcium sulfate	$CaSO_4$
Magnesium bicarbonate	$MgH_2(CO_3)_2$
Magnesium sulfate	$MgSO_4$
Sodium bicarbonate	$NaHCO_3$
Sodium carbonate	Na_2CO_3
Sodium chloride	NaCl
Sodium sulfate	Na_2SO_4

Calcium bicarbonate is popularly called lime when speaking of the mineral content of a water. The common names of sodium bicarbonate, sodium chloride, and sodium carbonate, are baking soda, common table salt, and sal soda, respectively. Sodium sulfate and magnesium sulfate in the crystalline form are known as Glaubers salts and Epsom salts respectively. Calcium sulfate in combination with water occurs in nature as the mineral gypsum. Collectively, most of the substances in the above list are popularly known as alkali salts. The sulfates of sodium, magnesium and calcium, and the chloride of sodium are classed as "white alkali," and sodium carbonate is known as "black alkali." A water solution of sodium carbonate is a solvent for the black humus of soil. On evaporation of a sodium carbonate solution that has been in contact with humus, a black residue is left. This explains the origin of the name "Black alkali." Sodium bicarbonate strictly speaking is not always classed as black alkali; when this salt

is introduced into the soil by irrigation water it may undergo chemical changes which form either sodium carbonate or sesquicarbonate. It follows that it is safer to include sodium bicarbonate as black alkali in determining the qualities of alkali water for irrigation.

It should be emphasized that a given ground water seldom contains in solution all the substances listed above. As a rule some one salt predominates, which imparts to the water the specific characteristics of the radicals which constitute the salt in question. A mixture of other salts in lesser and often times in negligible amounts, completes the mineral content of the water. On the other hand alkali waters of high concentration may contain excessive amounts of two or more salts. Even so the mineral characteristics of a given alkali water may usually be defined by specifying the radicals or salts that predominate.

On this basis it is possible to classify the mineral content of alkali waters occurring in Montana as follows:

(1) *Sulfate waters.* In this class the sulfate radical predominates. The predominant basic radicals which together have equivalent reacting values to the sulfate radical may be sodium, magnesium, and calcium. However, the predominant basic radicals in this class of ground waters are usually sodium and magnesium. The distinctive character of a water containing the sodium and sulphate radicals can be represented by the symbol $Na\text{-}SO_4$, which should not be confused with the chemical formula Na_2SO_4. A water containing magnesium in predominant amounts can be represented by the symbol $Mg\text{-}SO_4$, and a calcium water by the symbol $Ca\text{-}SO_4$.

A sodium sulfate water on evaporation to dryness yields crystals of the chemical salt popularly known as Glaubers salts. On the average, sodium sulfate forms between 60 and 70 percent of the total mineral salts in this class of waters. Magnesium sulfate or Epsom salts occurs much less frequently in Montana ground waters than does sodium sulfate or Glaubers salts.

(2) *Bicarbonate waters.* The predominant radical in this class is the acid radical known as bicarbonate, accompanied by the basic radicals sodium, calcium or magnesium. Sodium is usually the predominant basic radical in waters of this class which have a high mineral concentration. The symbol representing this type of mineral content would be $Na\text{-}HCO_3$. The sodium and bicarbonate radicals in combination form sodium bicarbonate, or baking soda. Sodium bicarbonate ground waters occur in a number of important sections of the state.

(3) *Chloride waters.* The sodium and chloride radicals are the predominant ones in this class. The symbol for the mineral content would be Na-Cl. On evaporation to dryness a water of this class would yield sodium chloride, or common table salt. Waters of this character are not encountered as frequently as waters of the other two classes, except in the deeper wells.

PHYSIOLOGIC EFFECTS

The rating of an alkali water for public consumption depends upon the mineral concentration of the water with respect to the physiologic reactions of the radicals that constitute the salts in solution. It has been previously explained that the radicals exist in solution as ions. The physiologic reactions of mineral waters therefore are not due to the chemical salts themselves, nor are they due to the elements of which the salts are composed. A rating standard can be formulated by considering the radicals that constitute the following salts which occur in greatest abundance in alkali waters, namely, sodium and magnesium sulfates, sodium carbonate, bicarbonate and chloride. When the physiologic reactions of the three classes of alkali waters described above having approximately the same mineral concentration, are considered separately, there is evidence to indicate that the radicals of bicarbonate waters have the most pronounced effect, radicals of chloride waters next, and the radicals of sulfate waters have the least effect when sodium is the basic radical and the magnesium radical is comparatively low.

The carbonate and bicarbonate radicals react with the hydrochloric acid of the stomach thus reducing the acidity of the gastric juice. Obviously an excessive alkalinity of the water ingested due to one or both of these radicals would cause digestive disturbances. However, the writer has on record information which indicates that certain individuals with abnormal stomach acidity have been benefited by changing to a drinking water of medium bicarbonate alkalinity.

In considering chloride waters it is generally considered that neither the sodium nor the chloride radicals, the predominant ones in this class, have very much specific action on body tissues, either as stimulants, depressants, or irritants. The physiologic effect of these radicals in a water ingested is due largely to what is known as a "salt action" in the intestines, as a result of which water is drawn into the intestinal tract diluting the contents of the intestine. Peristalsis is accelerated and therefore this salt in an alkali water has a laxative effect. However, it appears that sodium chloride solutions are ab-

sorbed from the alimentary canal, hence chloride waters do not have as much salt action as sulfate waters because sodium and magnesium sulfates are absorbed more slowly through the intestinal walls. It follows that sulfate waters would be more laxative than chloride waters.

Magnesium sulfate has the salt action referred to above, and the magnesium radical has a toxic effect in addition. It appears therefore that a water containing magnesium sulfate is more powerful in its action than a sodium sulfate water. This is further indicated by Reudeger's[1] investigations of North Dakota alkali waters, in which he compared the properties of ground waters of high mineral concentration containing varying amounts of sodium and magnesium sulfates. He discovered that alkali waters with the most objectionable properties contain considerable magnesium in the form of sulfate, and be reached the conclusion that a water containing over 200 parts per million of magnesium radical combined as sulfate is unsuitable for human consumption.

Hurlburt on the other hand, who has also studied the sulfate waters of North Dakota, states that 175 parts per million of magnesium is the upper limit that should be tolerated.

It is somewhat difficult to propose a standard for interpreting the qualities of an alkali water for public consumption. The problem is complicated by the fact that personal susceptibility to the physiologic reaction to mineral salts varies with the individual. On the other hand, experience indicates that the tolerance for dissolved mineral matter in drinking water is higher than generally believed, and it has been observed that mineral tolerance increases with the continued use of alkali waters for drinking.

After conducting an extended study of the therapeutic effect of dissolved mineral ingredients in drinking water the Ground Water Division of the U. S. Geological Survey has specified the concentration of the various radicals that would be unhealthful to most people. With slight modifications the Geological Survey standard is copied below:

	Parts per million
Bicarbonate radical (HCO_3)	700
Carbonate radical (CO_3)	350
Sulphate radical (SO_4)	1800
Chloride radical (Cl)	1500
Magnesium radical (Mg)	175

[1] Journal of American Public Health Association, 3: 10, pages 1094–99.

The data on page 1097 can be used as a general guide in rating a mineralized ground water for public consumption. There are so many variables to consider in the problem that each case should be thoroughly investigated. The allowable mineral concentration of a city water supply should naturally be below the limit that would cause marked physiologic disturbances in the case of visitors or transients unaccustomed to mineralized water. It is obvious that a higher mineral concentration can be approved for a private water supply which is used continuously by the same people, who thereby establish a high mineral tolerance. It follows, therefore, that approval of a mineralized water for a municipal supply should be based on a careful investigation.

HARDNESS

Natural waters containing the salts of calcium and magnesium in solution are classed as hard waters. Hardness is manifested by the action of the water on soap and by the formation of scale in steam boilers. There are two kinds of hardness. One is called temporary, because it can be removed by boiling, and is caused by calcium and magnesium bicarbonates. The second type is known as permanent hardness, which cannot be removed by boiling and is caused by calcium and magnesium sulfates in solution. The engineer who decides to deliver a softened water for public consumption can do so by installing a soda-lime water softening process, or he can use the Zeolite process of water softening. The hardness characteristic of some water would make it advisable to use Zeolitic softening in conjunction with the lime-soda process.

It is beyond the scope of this article to consider the methods of conditioning an alkali water for steam raising, and the attendant possibility of caustic embrittlement of steam boiler plate. The city engineer who is concerned with problems of this character should have a special investigation conducted to determine all the characteristics of the water from the standpoint of its industrial applications.

Iron and manganese compounds sometimes impart objectionable qualities to an alkali water which are apt to result in stained plumbing fixtures and fabrics in the laundry, and under some conditions a disagreeable taste. If the bad effects of iron are to be avoided the iron content must not exceed 0.2 p.p.m. Iron usually occurs in water in solution as ferrous bicarbonate. Iron can be readily removed by aëration, by the use of spray nozzles, by forcing air through the water

with diffusion aërators, and by the design of cascade aërators where the water falls over steps. By means of aëration iron is precipitated as oxide, which is insoluble. The insoluble iron oxide can then be removed by sedimentation or filtration, or by a combination of both procedures.

Construction methods and procedures can be specified that will prevent the chemical action of the salts of alkali water on the cement of a concrete structure. However, the specifications must be prepared in the light of the composition of set cement and the possible effects of alkali salts in water that may come in contact with structures of concrete. The following facts and theories form a basis for engineering practice in this case.

In the process of setting, tri-calcium silicate, one of the important components of cement, undergoes both hydrolysis and hydration, according to the reaction:

$$(CaO)_3SiO_2 + \text{Water} = CaO\cdot SiO_2\cdot 2\tfrac{1}{2}H_2O + 2Ca(OH)_2$$

The calcium hydroxide ($CaOH_2$) formed in this reaction serves an important function as binding material in set cement. By laboratory tests it has been shown that sulfates in solution react with calcium hydroxide ($CaOH_2$) and convert it into calcium sulfate in a form called gypsum, according to the following reaction:

$$Ca(OH)_2 + Na_2SO_4 + 2H_2O = 2NaOH + CaSO_4\cdot 2H_2O$$

Burke and Pinkney[2] have investigated this reaction by laboratory experiments in which solutions of alkali salts were forced by suction through set cement. They report that the calcium sulfate ($CaSO_4\cdot 2H_2O$) formed in the above reaction occupies a larger volume than the calcium hydroxide from which the same was formed. Therefore, it was concluded that sulfates in solution percolating through set cement alter the calcium hydroxide, the binding material, and convert it into a new substance which has a larger volume. If this reaction is permitted to take place it will have a disintegrating effect on concrete.

It follows therefore that the specifications for concrete must be such as to prevent the solutions of sulfates penetrating into the interior of a concrete structure. To accomplish this the engineer prepares a concrete mixture that is highly impervious to water. In other words adequate waterproofing will prevent this reaction from taking

[2] Bulletin 81, Montana Experiment Station.

place. In speaking of this point Leighou says that, "A great deal can be done towards making concrete waterproof by merely having the ingredients of the concrete rightly proportioned so that the voids are eliminated. It is said that concrete can be made entirely waterproof in this manner." On the other hand the engineer can also resort to the use of waterproofing compounds designed to exclude water from the structures. By adjusting his engineering practice and procedures in harmony with the chemistry of set cement and alkali water the engineer can solve this problem. The scope of this paper limits this discussion to mentioning the major problems that arise when alkali water comes in contact with concrete structures.

A municipal water supply usually furnishes water for the irrigation of lawns and gardens. The city engineer should investigate the effect of alkali salts on the soil and on plant growth. There is no standard of water quality for irrigation water containing alkali salts. Therefore, the engineer should consult a soil expert who can recognize and give due weight to the numerous variables that influence the effects of alkali salts in irrigation water. Some of the alkali salts tend to deflocculate the clay of soil and may produce an undesirable soil texture. At least one of the alkali salts, sodium carbonate, has a corrosive effect on plants and therefore a comparatively small amount of this salt can be tolerated in the soil solution. On the other hand, all alkali salts in the soil solution interfere in certain concentrations with the normal reactions of plant growth by varying the process of osmosis. The assimilation of plant food from the soil solution and the amount of water that the plant itself can absorb are related to the osmotic pressure relations in the plant juices and in the soil solution. There are other important variables to consider in rating an alkali water for irrigation. These variables are the amount of irrigation water used, the concentration of the mineral salts in solution, the physical character and chemical composition of the soil, and the depth of the soil with respect to under drainage.

(Presented before the Montana Section meeting April 21, 1934.)

William Gore

Died June 7, 1934

William Gore was born at King's Lynn, Norfolk, England, on April 13, 1871, the son of John and Mary Ann (Barrett) Gore. King's Lynn is situated in the Fenn country, a district largely below sea level and, like Holland, has been reclaimed from the sea by immense sea walls and dykes. In this district the rivers at high tide are below sea level and consequently have to be kept from overflowing their banks by a system of retaining walls. Throughout the district there is a great system of pumps and wind-mills to elevate the water into the rivers, where at low tide the pressure is great enough to force open the tidal gates and allow the rivers to discharge their accumulated waters into the sea. It was in such an atmosphere of irrigation, dykes, pumps, sea walls and every conceivable problem concerning water, with its control and management, that William Gore was raised.

He was educated at King's Lynn Technical School and the City and Guilds Institute of London, England. From 1888 to 1893 he was with the Highgate Iron Works and afterwards became Chief Draftsman to the Western Electric Company at Woolwich. In 1897 he joined the staff of the late George F. Deacon, LL.D., Civil Engineer and Water Works Expert. On the death of Dr. Deacon, in 1909, the business was taken over by Sir Alexander Binnie, Sons and Deacon and from then until 1912 Mr. Gore was Chief Assistant in the firm, carrying out large works of water supply, sewerage, irrigation and hydro-electric power in London, Liverpool, Birkenhead, Merthyr-Tydfil, Petrograd, Genoa, Athens, Malta and various places in India and Australia. Much of the work carried out at that time was connected with the design and construction of masonry dams, on which Mr. Gore wrote extensively. In 1912 he became Consulting Engineer to the Ransome-verMehr Machinery Company of London, England, in the development of the Ransome Drifting Sand Filter, of which system Mr. Gore was the inventor.

In 1913 Mr. Gore came to Canada and installed this system of water purification for the City of Toronto, the plant having a ca-

pacity of 60 million imperial gallons per day. In 1915 Mr. Gore developed a type of portable filter for water purification purposes which was used by the British, French and American armies.

In 1919 Mr. Gore formed a partnership with Colonel George G. Nasmith and William Storrie, under the name of Gore, Nasmith & Storrie, of Toronto, Canada, and this partnership continued until the time of his death. During his 21 years sojourn in Canada his firm carried out water supply and sewage disposal projects in Toronto, Ottawa, Windsor, Calgary, Belleville, Kitchener, Hamilton, St. Thomas, Oshawa, York Township, Port Hope and elsewhere.

Mr. Gore's experience in dealing with the problems in these municipalities covered a very wide field. He had the inventive type of brain and beside the drifting sand filter he has to his credit what is known as the "Gore Mixing Chamber" in water purification systems. This method of mixing chemicals with the water was first worked out in the experimental plant at Ottawa and the results obtained in several of the filter plants since installed are somewhat remarkable.

In 1893 Mr. Gore won the Norfolk County Scholarship; in 1894 the Whitworth Scholarship; in 1896 the Siemen Memorial Medal; and in 1907 the George Stephenson Gold Medal for research work.

In 1897 Mr. Gore was married to Kate Daisley, who survives him, along with one son, George Gore, with the Canadian General Electric Company of Peterboro, Ontario.

Mr. Gore was a Fellow of the City and Guilds Institute of London, England; a Member of the Institution of Civil Engineers, London, England; a Member of the American Society of Civil Engineers; a Member of the Engineering Institute of Canada; a Member of the American Water Works Association and other technical societies.

Mr. Gore's varied professional experience, his wide knowledge of scientific subjects, his extraordinary memory for facts and details, his keen insight and knowledge of construction work, made his advice thoroughly reliable on all engineering matters. But above all his extreme modesty and gentleness made him a great favorite with all who came in contact with him, and his willingness to assist and give advice to the younger members of his profession was appreciated by one and all.

It seemed somewhat fitting that Mother Nature should pay her tribute to one who had spent so much of his life dealing with the forces in nature for the benefit and convenience of mankind for his mortal remains were placed in the grave accompanied by a sudden and heavy shower of rain.

ABSTRACTS OF WATER WORKS LITERATURE

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Ground and Surface Water Rights Cleared by Decision. THOMAS H. MEANS. Eng. News-Rec., 108: 126, January 28, 1932. Settling an important point in water law, Arizona Supreme Court has reversed lower court decision and definitely established doctrine that ground water is to be considered as percolating unless proved to flow in channels "by clear and convincing evidence." Since Arizona law holds that only water flowing in channels is subject to appropriation, decision assures developer of surface waters against interference by claims to subterranean waters established by inference and by doubtful evidence as to presence of underground channels. Suit was brought by Southwestern Cotton Company against Maricopa County Municipal Water-Conservation District. Litigation arose over construction of Frog Tanks, or Lake Pleasant storage reservoir, on Agua Fria River about 30 miles northwest of Phoenix. River drains 1500 square miles of mountains, and at point where it enters plain, or valley, it has no sustained flow; its floods are very irregular and are separated by long periods when wide sandy bed is dry. Southwestern Cotton Company owned lands on both sides of river below dam and had bored a number of wells and used the water for irrigation. Of these wells, a few were close to, or in river bed, but most of them were scattered over valley, miles from river. Company claimed that, by prior appropriation through its wells, it had acquired right to floods to extent of its irrigation requirements, and contended that waters sinking in Agua Fria sandy bed fed wells directly and that there were known underground channels from river to wells. Opposing theory was based upon geological history of valley, as evidenced by present physiographic features, contending that whole area is heterogeneous deposit without regularity of stratification and with no known stringers of material capable of forming continuous aquifers. Wells in almost any place in such a débris fan will encounter water, and whole body of ground water takes on all characteristics of body of percolating water slowly moving toward outlet of valley rather than moving in series of underground channels. Court concludes, "There is not a scintilla of evidence in the record from which the ordinary man, or even the trained scientist, could point out definitely a specific place where any one of the so-called subterranean water courses begins, where it ends, or how far its banks extend." Summary of probable effect of decision is included.— *R. E. Thompson.*

Progress on New Mexico Control of Underground Water Supplies. Eng. News-Rec., **108**: 325, March 3, 1932. Through correction of legal flaw in original act, making underground waters subject to appropriation, and provision of $20,000 for plugging leaking wells in Roswell artesian basin, New Mexico is making definite progress in control of ground water supplies. Object of law is to place "the waters of underground streams, channels, artesian basins, reservoirs or lakes, having reasonably ascertainable boundaries" under jurisdiction of state and declare them public waters. Such declaration makes them subject to appropriation and to rules of prior rights, beneficial use, and other features of appropriation doctrine. Rights to use are to be established in manner paralleling appropriation and use of surface waters. Application to state engineer is required and permits for use are necessary. Study indicated that many of older wells in Roswell basin area were badly in need of repair, rusting of casing in many cases permitting artesian supply to leak into upper strata. Methods of plugging with cement, commonly used in oil fields, are not economically feasible, and study is being made along lines of mud plugging, using concrete cap to maintain plug in position.—*R. E. Thompson.*

Long Earth Cores Obtained by New Driven-Casing Method. Eng. News-Rec., **108**: 324, March 3, 1932. Difficult task of obtaining intact earth cores 30 to 40 feet long has been successfully performed by newly developed method at West Side sewage treatment plant of Sanitary District of Chicago. Method consists of driving successively to desired depth two structural-steel shapes, a 4 x 4-inch angle and a 7-inch channel, and withdrawing them simultaneously to inclose triangular core. Procedure was developed by A. J. Forschner and R. H. Burke of T. J. Forschner Construction Company, which has patented the method. Air-driven pile hammer is employed for driving, and withdrawal is effected with crane. Under no condition is there any slippage at lower end of core.—*R. E. Thompson.*

Silt-Sampling Apparatus Used on the Missouri River. R. Whitaker. Eng. News-Rec., **108**: 395, March 17, 1932. Silt Section of United States Engineer Office at Kansas City, Mo., has used two types of samplers for suspended sediment and two types of apparatus for obtaining samples of bed sediment. The Au type United States Geological Survey silt sampler, used throughout 1929, consists of frame that holds a pint milk bottle vertically. After lowering to desired depth, a small drop-weight is allowed to slide down suspending cable and to strike a plunger, which drives knife-edge cutter through paper cap sealing bottle and allows water to enter. Float valve under cap seals bottle when about 400 cc. has entered. This sampler has been discarded in favor of Straub silt sampler, developed by silt section of the office. Straub sampler is development of Au type. It holds capped milk bottle vertically, cutting plunger being held 1 inch above cap by vertical spring. Small drop-weight is allowed to slide down suspending cable, forcing cutter through cap. Spring then lifts drop-weight and cutting plunger clear of perforated cap, so that water can enter without obstruction. Small hand-operated orange-peel bucket, 220 cubic inches capacity, was used in 1929 to obtain specimens of bed sediment. This apparatus, however, did not prove very satisfactory as launch and large

party, and also considerable time, were required to operate it. LUGN cylindrical bed-sediment sampler has been used since November, 1929. It consists of cylinder that seats on circular base, with stem extending from base through cylinder. When sampler is dragged along river bottom from bridge, or from small boat by means of rope attached to stem, cylinder tilts down and cuts into bottom material but seats on base again and seals tightly when raised to surface. This sampler is light and positive in action, and has proved very satisfactory.—*R. E. Thompson.*

Stabilization of Alluvial Rivers in India. GORDON HEARN. Eng. News-Rec., 108: 393–5, March 17, 1932. Author outlines and discusses problems involved in checking meanderings of rivers in India, as related to Mississippi problem.—*R. E. Thompson.*

Tests on Activated Carbon Show Its Selective Nature. MORTIMER M. GIBBONS. Eng. News-Rec., 108: 391–2, March 17, 1932. Water supply of Rahway, N. J., derived from Rahway River, is frequently contaminated with wastes from manufacture of organic chemicals, which cause disagreeable tastes and odors. In addition, algae, particularly *Synura*, occasionally give rise to tastes and odors. Purification process consists of coagulation with alum, preliminary aëration, sedimentation (6 hours), filtration, chlorination, and secondary aëration. During 1930 and 1931, experiments were carried out with activated carbon, both granular and powdered, as remedy for these objectionable conditions. Experiments on upward and downward filtration, through granular activated carbon were conducted in gravity filter, 16 inches in diameter, containing 24 inches of carbon of effective size 0.74 mm. and uniformity coefficient 3.3. Filtration rate of 4 gallons per square foot per minute gave satisfactory odor removal, but was too high to effect complete decolorization and dechlorination. Increasing rate to 5 gallons per square foot per minute decreased odor removal. Adsorptive properties of carbon appeared to be somewhat reduced after 8 months' service. Action of carbon was found to be selective, being particularly successful with medicinal odors. Bitter odors, due to fermenting wastes, were only slightly reduced, and cucumber odor, due to *Synura*, though somewhat reduced, remained strong enough to be objectionable. Application of 3.5 p.p.m. of powdered activated carbon on plant scale at alum-application point (providing 6 hours contact) slightly reduced cucumber odor, but failed to render effluent unobjectionable. Application of 0.5–2.0 p.p.m. carbon by means of inductor into filter influent conduit (20 minutes contact) eliminated odors due to vegetable growths and also woody taste, believed to be due to chlorinated organic matter; but increase in filter effluent turbidity occurred, probably due to agitation of flocculated water immediately prior to filtration. Changing point of application to entrance to secondary coagulation basin, which provides contact period of 3 hours, corrected this difficulty and this treatment has been regularly employed since that time (spring of 1931), dosage used varying from 1 to 8 p.p.m. and averaging 2 p.p.m. Vegetable odors, except those due to *Synura*, were successfully eliminated. Preliminary chlorination in conjunction with carbon, however, completely removed cucumber taste. Filter runs have not been appreciably short-

ened except when more than 5 p.p.m. of carbon has been applied. With high dosage, filter runs were reduced as much as 50 per cent. Carbon is not completely washed out of sand in wash period of 5 minutes at velocity of 24 inches per minute, and when it has been necessary to apply large amounts of carbon for several days in succession, increase of nearly 1 foot in initial loss of head has been observed. The carbon has penetrated sand bed to depth of 6 inches in 8 months. To remedy this condition, it will be necessary to wash top 6 inches of sand in sand-washer at least once each year.—*R. E. Thompson.*

Progress on Toronto's New Waterworks. R. C. Harris. Cont. Rec. and Eng. Rev., **46**: 97–8, February 3, 1932. Brief statement of progress to date. The St. Clair distribution reservoir (50 million gallons) has been completed and will be placed in service in near future. Rock excavation for intake and filtered water tunnels (6 to 10 feet in diameter) has been completed and approximately 30 per cent of concrete lining has been placed. The 6-foot section of tunnel is 18,575 feet long; 7-foot section, 31,093 feet; and 10-foot section, 3,270 feet. Fabrication of pipe for intake is underway and actual construction will be commenced as early in spring as weather will permit. Tenders for construction of Victoria Park filtration plant and works incidental thereto will be advertised for an early date and plans are being prepared for 3 new pumping stations and for elevated water tower. Good progress has been made on new large mains made necessary by construction of new supply works.—*R. E. Thompson.*

Absolute Volume Proportioning of Concrete. Joseph A. Kitts. Eng. News-Rec., **108**: 332, 1932. One great advantage of cement-water by weight strength relation of Lyse (Cf. C. A., **26**: 575) is, that it really measures effect of absolute volume of cement in unit volume of water. It will be found that adoption of absolute volume as basic measure for characteristics of materials and for proportions of mixtures simplifies whole subject of physics and technology of concrete mixtures, gives simple and convenient methods of analysis and calculation, and is only available basis for predetermination of yield, density, mortar content, etc.—*R. E. Thompson (Courtesy Chem. Abst.).*

Large Steel-Lined Tunnel Shafts Sunk at High Speed. Eng. News-Rec., **108**: 317, March 3, 1932. Ordinary troubles in shaft sinking in soft ground, particularly where quicksand is penetrated, were avoided with remarkable success in sinking 6 working shafts for sewer-tunnel construction in Cleveland, Ohio. Flanged plates were used for lining shafts, without timbering or braces. Plates, 16 x 37$\frac{11}{16}$ inches, were set in place one by one, forming rings, and proceeding from top downward, with so little exposure of unsupported shaft wall at any one time that sloughing was easily controlled. Shafts were of unusual diameter (16 feet) considering their moderate depth of 60 feet and size of tunnel (5-foot, brick-lined). One shaft was excavated and lined in 192 man-hours, and others in comparably short times.—*R. E. Thompson.*

Canal-Bank Maintenance by Disking on Imperial Irrigation District. M. J. Dowd. Eng. News-Rec., **108**: 206–7, February 11, 1932. Use of tractor-

drawn disk has reduced cost of canal-bank maintenance on Imperial Irrigation District works by about one-half as against hand labor. There is also decided tendency toward thinning out of re-growth as compared with hand-cutting.—*R. E. Thompson.*

Use Cement-Water Ratio Judiciously. GEORGE A. SMITH. Eng. News-Rec., **108**: 298, 1932. S. believes that *c/w* strength relation, as proposed by LYSE (C. A., **26**: 575), will be found an expeditious tool for most practical conditions where variables extend over relatively narrow range; but that in case of relatively wide variations, *c/w* straight-line relation will show deviation from actual conditions.—*R. E. Thompson (Courtesy Chem. Abst.).*

Waterworks Progress in Kitchener, Ont. MARCEL PEQUEGNAT. Cont. Rec. and Eng. Rev., **46**: 98–9, February 3, 1932. Progress in Kitchener in regard to water supply during 1931 is reviewed. Consumption averaged somewhat in excess of 2.5 m.g.d. (82 gallons per capita), decrease of 6.5 percent compared with previous year. New Kelly gravel-wall well was constructed, but owing to decreased yield of other wells, in which water level has been slowly receding, available supply of approximately 5 m.g.d. was only slightly increased. Statement of chemical composition of water is included.—*R. E. Thompson.*

Knock-Down Suction Dredge Cleans Reservoir. Eng. News-Rec., **108**: 329–30, March 3, 1932. Suction dredge with cutter machinery, acquired by Newark, N. J., department of public works for removing silt deposits from reservoirs, is said to be smallest ever built. First task was cleaning Macopin intake, 32-million gallon reservoir, with surface area of 12 acres, used as collecting and settling basin on Pequannock watershed. Deposits, varying from thin film to depth of 4 feet, had been causing turbidity and pipe deposits. As material was too inert to remove by straight suction methods, use of suction and cutter dredge, supplemented by hydraulic equipment, was decided upon. Ease of dismantling was factor in choice of equipment. Hull is made in two pontoon sections, to be bolted together in water, and is light enough for truck transportation.—*R. E. Thompson.*

Simplifying Design and Control of Concrete Mixes. INGE LYSE. Eng. News-Rec., **108**: 248–9, 1932. Method of designing concrete mixes adopted for graduate course in plain concrete at Lehigh University, which involves cement-water ratio theory discussed by L. (C. A., **26**: 575), is described.—*R. E. Thompson (Courtesy Chem. Abst.).*

Immense New Water Supply for Paris Authorized. Eng. News-Rec., **108**: 316, March 3, 1932. Infiltration galleries in Loire Valley to yield more than 0.25 billion gallons of water daily and 14¾-foot gravity-flow aqueduct, 90 miles long, are projected to provide new supply for Paris. Estimated cost is about 2 billion francs ($80,000,000). New supply will supplement the "spring water" with which two-thirds of city is now supplied, will displace filtered and disinfected river water that makes up balance of present supply, and will possibly serve some of suburbs. Hardness of new supply, it is expected, will be only

12°, as compared with 25 to 30° of Seine supply. Filter galleries will be located in homogeneous sandy deposit 0.32 mile wide and 43.5 miles long lying on left side of Loire at elevation such as to make gravity delivery possible. Infiltration system will consist of central collecting conduit, 11.5 feet in diameter at lower end, fed by galleries, main conduit being from 27 to 34 feet below ground surface. Water will probably not require disinfection. Aqueduct, where under gravity flow, will be of masonry, 14.76 feet in diameter; where under pressure (inverted siphons) two 9.72-foot reinforced-concrete conduits will be employed. Combined length of main filtration gallery and aqueduct will be 133.5 miles, terminating in reservoir.—*R. E. Thompson.*

Engineers Named to Investigate Baltimore's Water Supply. Eng. News-Rec., **108**: 194, February 4, 1932. Board of Estimates, Baltimore, has approved appointment by Public Improvement Commission of John H. Gregory, J. G. Requardt, and Abel Wolman to make study for plans for additional water supply for city. The engineers will be given free rein in making their investigations and recommendations.—*R. E. Thompson.*

Sealing a Leaky Reservoir in Italian Appenines. B. C. Collier. Eng. News-Rec., **108**: 293–4, February 25, 1932. Brief details given from article by Mario Mortara and Vincenzo Buttiglione in Annali dei Lavori Pubblici, Gia Giornale del Genis Civile, official gazette of civil engineering service of Italian Government. In 1914 work was begun on dam across canyon of River Ripa near town of Muro, in mountains of south central Italy. Dam was competed in 1916, but leakage through walls of canyon prevented operation and created dangerous condition. Dam has gravity-arch section about 30 meters long and 47 meters high above stream bed. Examination showed that while some loss occurred through crevasses, principal loss was through numerous laminated seams in rock. Repair work consisted of filling crevasses with concrete and covering whole surface of reservoir with two layers of gunite, each about 2 centimeters thick. After many difficulties, work was finally completed in 1929, repair cost totaling about $308,000. Total surface of gunite was 39,000 square meters.—*R. E. Thompson.*

Planning Water and Sewer Systems for a Large Industrial Plant. E. D. George and F. W. Buck. Eng. News-Rec., **108**: 120–2, January 28, 1932. Description of water and sewer systems of Point Breeze (Baltimore) plant of Western Electric Company. Three separate water systems were provided, sources being city mains, Patapsco River, and driven wells. For drinking purposes, city water is chilled in large tank by means of 100-ton refrigerating machine and circulated by centrifugal pumps. Make-up water is filtered and ozonated, provision also being made for ozonation (for part of day) of return water. Wells were constructed for supply water for purposes requiring cold water throughout year.—*R. E. Thompson.*

Steel Sheet-piling Carries Tunnel Through Fault Zone. Eng. News-Rec., **108**: 361–4, March 10, 1932. Shield of steel sheet-piling driven horizontally, first for top-center drift and later around perimenter of full-section enlarge-

ment (except bottom segment) proved successful in carrying New York water tunnel No. 2 through short fault zone under Bronx River. Driving of 21-foot tunnel was interrupted for 6 months in effort to progress 50 feet through heavy water-bearing section of disintegrated rock lying between limestone and gneiss formations, material that, when once confined, developed hydrostatic head of 250 pounds per square inch. Tunnel, extending for 20 miles from Hill View Reservoir in Yonkers to Fort Hamilton Park in Brooklyn, is for most part circular in section, with diameter of 17 feet inside concrete lining and 21 feet inside rock, and is being driven from 19 shafts. Details are given of geology of river crossing and of elaborate grouting schedule, employing pressures as high as 700 pounds and requiring thousands of bags of cement, which checked rush of mud and water into tunnel. Special lining is now being placed that will serve to carry tunnel across fault, acting as beam over soft ground. Although section of tunnel had been enlarged to diameter of 24 feet upon approaching fault zone, to allow for increased thickness of lining, necessity of driving sheeting within already-erected steel timbering reduced ultimate inside diameter to 14 feet. Space between sheeting and outer steel lining was filled with grout, and 27-inch thickness of heavily reinforced concrete was placed inside sheeting. Inside of this concrete lining will be placed a steel pipe 16.5 feet in diameter, built of 1-inch boiler plate. The sections, 46.5 inches long, will be flange-connected by angles placed inside shell, and will be fabricated erect in chamber where main bulkhead used in stopping flow of mud and water now stands. Several sections will be built up and skidded into place over rails laid in concrete invert. The 6-inch clearance between concrete and pipe will be grouted under high pressure and 14-inch lining of unreinforced concrete will be placed inside pipe.—*R. E. Thompson.*

Developing Water Power on the Mokelumne River. Eng. News-Rec., **108**: 350-3, March 10, 1932. Two plants (of 71,000-kva. combined capacity) of Mokelumne River hydro-electric development program of Pacific Gas and Electric Company are now in operation and completion of remainder of project is scheduled for 1934. Salt Springs dam and power plant, conduit to Tiger Creek plant, and latter plant itself have been completed. Remaining features of program in order of development are: (1) reconstruction of Electra plant; (2) installation of 15,000-kva. West Point plant between Tiger Creek and Electra; and (3) diversion of Bear River flow through tunnel to Salt Springs plant, whence it will flow through lower plants. Salt Springs dam has been previously described. Discharge from Salt Springs plant flows directly into Tiger Creek conduit, which extends 21 miles to forebay above Tiger Creek power house. With exception of 2.5 miles of tunnel and 2 steel siphons, conduit consists of concrete flume of 550 and 625-second-foot capacity located mainly on excavated bench. Several intermediate streams are directed into conduit. Use of steel panel forms, moved and set by motorized handling equipment, and mobile concreting plants characterized construction lay-out. Flume is 7 feet high by 14 feet 3 inches wide and follows falling contour around side of mountain. About 2.5 miles above forebay, regulating reservoir, formed by buttress-and-slab type concrete dam 100 feet high, was constructed on Tiger Creek watershed to impound flow of that stream and control flow in conduit.

Forebay is artificial basin made by excavating 200,000 cubic yards of material from ridge above power house, sides and bottom being covered with 2 inches of gunite.—*R. E. Thompson.*

Stability of Dams Increased by More Economical Use of Materials. Calvin V. Davis. Eng. News-Rec., **108**: 210–4, February 11, 1932. Discussion of design of concrete dams in which it is pointed out that obvious solution of problem of obtaining both increased safety and economy of construction is by elimination of surplus of useless material in conventional gravity dam by coring out of massive section. This procedure logically causes gravity section to evolve to buttress type. Improved designs thus obtained have safety factors comparable with those used for other important engineering structures.—*R. E. Thompson.*

Economies in Dams Designed for Increase in Height. Calvin V. Davis. Eng. News-Rec., **108**: 292–3, February 25, 1932. Completion of work of adding 30 feet to height of Bristol dam of Utilities Power Company on Pemigewasset River in New Hampshire emphasizes economic possibilities of providing for subsequent increase in height when dam first built. Dam was built in 1924 to height sufficient to develop head of 55 feet with 5-foot flash-boards. Application of this principle to design of many dams presents possibilities for major economies when expansion of service, either power or water supply, becomes desirable. Application of principal stress analysis to design of dam which is to be raised at some future time is discussed.—*R. E. Thompson.*

New Form of Dam Proposed for Narrow Rock Canyons. C. H. Howell. Eng. News-Rec., **108**: 214–6, February 11, 1932. Dam in which water pressure is transmitted direct to abutments by thrust walls, or buttresses, instead of to valley bottom, can neither overturn nor slide. In suitable design based on this principle, effect of uplift is nullified. Temperature and volume changes of concrete do not affect maximum stresses and, with assumption of unyielding abutments, concrete stresses are determinate and not excessive. No tensile stresses occur and dam is economical. The new type, called thrust-buttress dam, is suited only to narrow canyons with sound rock abutments. It contains from 40 to 60 per cent of concrete required for gravity dam. Saving in river bed excavation and in concrete volume below river bed will generally be even greater. Partly off-setting these advantages, thrust-buttress dam will require somewhat more abutment excavation. Compared with arch dam, this type will probably show saving only at sites exceptionally favorable for thrust buttresses. Fundamental idea of dividing dam into vertical units, as in thrust-buttress design, can also be applied to gravity dams, with marked economy over usual gravity layout.—*R. E. Thompson.*

Erosion Below Conowingo Dam Proves Value of Model Tests. L. N. Reeve. Eng. News-Rec., **108**: 127–30, January 28, 1932. Details given of model tests made in connection with design of Philadelphia Electric Company's Conowingo dam, on Susquehanna River in Maryland. Examination of river bed clearly indicated that model tests had accurately predicted where and how

erosion would develop below dam, and emphasizes value of such studies in determining design that will give maximum safety and economy.—*R. E. Thompson.*

Depressed Apron Checks Erosion Below Hamilton Dam. C. H. Eiffert. Eng. News-Rec., **108**: 130–2, January 28, 1932. Flexible apron of concrete blocks, downstream from monolithic apron of Hamilton overflow dam, of Miami Conservancy District on Great Miami River, proved unsatisfactory. Provision has now been made for dissipation of energy in pool over depressed apron. Dam prevents large amounts of sand and gravel from being carried into improved river channel, two screening and washing plants having been provided for handling gravel that accumulates.—*R. E. Thompson.*

Minor Repairs Restore Usefulness of Earth Dam in Arizona. Eng. News-Rec., **108**: 356–7, March 10, 1932. Repair of Lyman dam, earth structure for irrigation on Little Colorado River, and enlargement of spillway to insure against overtopping have recently been completed. Dam was built in 1913 and failed in 1915 as result of rapid filling of reservoir (7.5 billion gallons) during storm. As rebuilt, dam was 800 feet long and 65 feet high (maximum). Repairs consisted of raising crest about 10 inches to uniform level and adding earth on downstream slope to maintain original crest width of 8 feet and fill gullies that had been formed by rains. As spillway capacity was only 33 per cent of maximum expected flood, it was widened to full capacity. As additional factor of safety, freeboard of 5 feet above maximum flood level was provided. Lining of outlet tunnel with gunite effectively stopped leakage through concrete.—*R. E. Thompson.*

New Construction Tempo Being Set at Hoover Dam. Eng. News-Rec., **108**: 179–80, February 4, 1932. Brief outline of progress on Hoover dam project. All former tunneling records have been broken in consistent progress of from 230 to 260 feet per day that is being made in driving eight 41 × 56-foot enlarged headings in the 4 long diversion tunnels. With 20-foot drill holes, 15 feet of heading is being pulled with each shot. Using three 8-hour shifts, average of more than 2 rounds per day is being maintained in each heading.—*R. E. Thompson.*

First Major Low-Head Power Plant in the West: Rock Island. W. D. Shannon. Eng. News-Rec., **108**: 240–4, February 18, 1932. Detailed, illustrated description of Rock Island development of Puget Sound Power and Light Company on Columbia River. Ultimate 250,000-horsepower installation will use 54,000 second-feet of water at maximum head of 51 feet. Initial 84,000-horsepower installation will be completed in 1932.—*R. E. Thompson.*

Mississippi River Work Ahead of Schedule. T. H. Jackson. Eng. News-Rec., **108**: 178–9, February 4, 1932. Progress on Mississippi River flood protection project during 1931 is reviewed. To December 31, about $104,500,000 had been expended under flood control act of May 15, 1928. New record in

levee construction was established, 93.9 million cubic yards having been placed during year.—*R. E. Thompson.*

The Effect of Acid Waters on Concrete. BAILEY TREMPER. J. Am. Concrete Inst., 3: 1–32, 1931. From Chem. Abst., 26: 1410, March 10, 1932. In waters ranging in pH from 6.0 to 7.0, degree of attack is inversely proportional to pH value; attack is negligible in waters of pH greater than 7.0. Brand of cement, character of aggregate, and temperature of curing (below 100°) have little effect on resistance. High-quality concrete is markedly more resistant than poor concrete. When about half original lime (of cement) is dissolved from average concrete, complete loss of strength and coherence results.—*R. E. Thompson.*

Methods for Determination of Low Concentrations of Chlorine. V. N. KOLUICHEVA and R. V. TEIS. J. Russ. Phys.-Chem. Soc., 62: 1957–73, 1930. From Chem. Abst., 26: 1211, March 10, 1932. Various methods for determining small amounts of chlorine are reviewed. Potentiometric method was found to be most accurate, but for purposes of water purification control, colorimetric method is preferable, because of simplicity.—*R. E. Thompson.*

Choice of an International Index for the Examination of Water. A. SULFRIAN. Ann. chim. anal. chim. appl., 13: 353–6, 1931. From Chem. Abst., 26: 1364, March 10, 1932. To avoid confusion, it would be better for all countries to use same unit; use of milliequivalent per liter is proposed. Advantages are pointed out.—*R. E. Thompson.*

Lead Poisoning from the Use of Tap Water. F. TÖDT. Centr. Gesundh. u. Städtehyg., 1930: 2–4; Wasser u. Abwasser, 28: 98. From Chem. Abst., 26: 1366, March 10, 1932. In Prussia, lead content of 0.3 p.p.m. is allowed. This limit is too high. Lead pipe should be avoided; it may be replaced by copper, or by asphalt-coated iron, pipes. Also in Apparatebau, 42: 258–60, 1930.—*R. E. Thompson.*

Apparatus for Taking Water Samples from Different Levels. J. ARTHUR REYNIERS. Science, 75: 83–4, 1932. From Chem. Abst., 26: 1365, March 10, 1932.—*R. E. Thompson.*

Colloid Theory of the Corrosion of Iron and Steel. J. A. NEWTON FRIEND. Trans. Faraday Soc., 27: 595–6, 1931. From Chem. Abst., 26: 1224, March 10, 1932. Author refers to previous experiments (C. A., 15: 2360) on effect of increasing velocity of ordinary tap water relative to surface of iron. Later work with acid solutions showed that rate of solution of iron was directly proportional to velocity of rotation when certain iron disks were caused to rotate at various speeds in glass tanks containing dilute sulfuric acid. FRIEND concluded that a great difference existed between corrosion in more or less neutral solutions and corrosion due to acid attack. This view led to new theory of corrosion, which assumed iron to be noble, or passive, to pure aërated water and only to pass into solution very slowly in presence of catalyst, dissolved iron

probably being present at first in solution in more or less completely ionized condition; agglomeration of ferrous hydroxide then takes place, colloid thus catalytically formed assisting the as yet unattacked metal to undergo corrosion. Series of carefully planned experiments proved that action of this colloid cannot be chemical. It is probably simply mechanical; i.e., in rapidly moving aërated water iron passes very slowly into solution and hydroxide is swept away, whereas under stationary or slow-moving conditions iron passes into solution as before, but hydroxide agglomerates to sol or gel and is adsorbed on, or otherwise clings to, portions of metallic surface, thus partially screening them from oxygen and inducing anodic corrosion.—*R. E. Thompson.*

The Influence of pH on the Presence of Bacteriophages in Industrial Water. P. Marginesu. Igiene mod., **1929**: 201–6; Wasser u. Abwasser, **28**: 24. From Chem. Abst., **26**: 1365, March 10, 1932. In alkaline water the phage against various pathogenic bacteria could always be detected. In water of pH 5.2, the phages for *B. coli* and *Staphylococcus* but not for *B. typhosus* or *dysenteriae* were found. In water of pH 3.5, no bacteriophages were found. Significance of pH of natural water on disappearance of phages was confirmed in laboratory. —*R. E. Thompson.*

The Study of a Bacteriophage Occurring in River Water. S. Predtecenskii. Zentr. ges. Hyg. Bakt. Immunitäts, **22**: 329–30, 1930; Wasser u. Abwasser, **27**: 259. From Chem. Abst., **26**: 1365, March 10, 1932. Author found phage in polluted river water that reacted strongly against *B. coli*, slightly against related strains, and not at all against other fecal bacteria. In bacteriological examination of water, possibility of this phage being present must be considered. It can cause disappearance of *B. coli* or produce atypical variations. —*R. E. Thompson.*

The Bacteriophage as a Measure of the Self-Purification Power of River Water. S. Segre. Zentr. ges. Hyg. Bakt. Immunitäts, **22**: 528, 1930. From Chem. Abst., **26**: 1365, March 10, 1932. Bacteriophages were isolated which had strong lytic action on Shiga, Flexner, cholera, typhoid, and colon bacteria. Bacteriophage was not very sensitive toward chemical impurities in industrial waters. Phage was still active after 24 hours at 37°.—*R. E. Thompson.*

Injury to Fish from Sugar Factory Waste with Special Attention to the Saponin Content of the Water. G. Ebeling. Z. Fischerei, **29**: 53, 1931; Wasser u. Abwasser, **28**: 256. From Chem. Abst., **26**: 1368, March 10, 1932. Acid saponin was injurious to fish in concentration of about 5 milligrams per liter. —*R. E. Thompson.*

Testing the Reaction of Distilled Water. Emil Truog. Science, **74**: 633–4, 1931. From Chem. Abst., **26**: 1485, March 20, 1932. pH of distilled water in equilibrium with atmosphere was found to be 5.6 to 5.8.—*R. E. Thompson.*

The Exchange of Bases in Permutite and Surface Adsorption by Silica Gels. I. R. Haas. Chem.-Ztg., **55**: 975–6, 1931. From Chem. Abst., **26**: 1497,

March 20, 1932. In permutite, exchange of bases depends upon ionic diffusion, but in silica gels, it depends upon adsorption. Structural formulas for permutite are discussed.—*R. E. Thompson.*

Shanghai Waterworks Extensions. H. STRINGER. Engineer, **152**: 648-50, 669-71, 1931. From Chem. Abst., **26**: 1684, March 20, 1932. Purification presents unusual difficulties because of high turbidity, 223, and high average bacterial count, 80,635, at 20°.—*R. E. Thompson.*

Analysis of Aluminum Sulfate. WILDER D. BANCROFT, HERBERT L. DAVIS and ESTHER C. FARNHAM. J. Phys. Chem., **36**: 515-22, 1932. From Chem. Abst., **26**: 1542, March 20, 1932. Bottle of hydrated aluminum sulfate crystals proved very uneven in composition, requiring about as much care to get representative sample as would mineral ores. Contrary to expectation, it was found easy to get good results in determining SO_3 content as barium sulfate, provided a little hydrochloric acid was added to prevent hydrolysis. Even when such precaution was not taken, maximum error was only 0.7 per cent of true content, whereas KÜSTER and THIEL got as much as 9.9 per cent error in analyzing ferric chloride-sulfuric acid solutions. It was also found possible to get good Al_2O_3 values by simply igniting the salt. At from 400° to 500°, moisture is lost without appreciable volatilization of SO_3; then, by igniting over blast lamp, all SO_3 can be removed.—*R. E. Thompson.*

Reduction of Nitrates by Es. coli. LEONARD HUBERT STRICKLAND. Biochem. J., **25**: 1543-54, 1931. From Chem. Abst., **26**: 1636, March 20, 1932. *Es. coli* reduces nitrate quantitatively to nitrite. After toluene treatment it oxidizes formate, lactate, and succinate to same degree at expense of nitrate as it does with oxygen, that is, to carbon dioxide, pyruvate, and fumarate, respectively. Reduction is inhibited by cyanide, but not by carbon monoxide. —*R. E. Thompson.*

Effect of Bacteriophage on the Oxidation-Reduction Potentials of E. Coli Cultures. LESLIE F. HEWITT. Biochem. J., **25**: 1641-6, 1931. From Chem. Abst., **26**: 1636, March 20, 1932. Effect of bacteriophage on oxidation-reduction potential follows effect on proliferation of bacteria, except that in aërobic glucose broth cultures of *E. coli* containing bacteriophage there is initial fall in potential without appreciable bacterial growth.—*R. E. Thompson.*

Continuous Control of the pH Value of Waters. W. KORDATSKI. Chem.-Ztg., **56**: 19-20, 1932. From Chem. Abst., **26**: 1685, March 20, 1932. Solution of quinhydrone in acetone is introduced proportionally into water to be measured, a mercury-controlled regulator actuated by stream of hydrant water serving as measuring mechanism. Solution so prepared flows into electrode vessel every 3 minutes, and pH developed therein is recorded automatically by suitable millivoltmeter.—*R. E. Thompson.*

Further Experience of Bismuth Sulfite Media in the Isolation of Bacillus Typhosus and Bacillus Paratyphosus B from Feces, Sewage, and Water. H.

JAMES WILSON and E. M. McV. BLAIR. J. Hyg., 31: 138–61, 1931. From Chem. Abst., 26: 1689, March 20, 1932.—*R. E. Thompson.*

Action on Concrete of Water Containing Carbon Dioxide. N. SUNDIUS. Beton u. Eisen, 1930: 2, 14 pp. From Chem. Abst., 26: 1714, March 20, 1932. Carbon dioxide content plays subordinate part in attack on pervious concrete. With dense concrete, rate of water flow is important; with low rate, high carbon dioxide content has little action.—*R. E. Thompson.*

Deposition of Manganese. CARL ZAPFFE. Econ. Geol., 26: 799–832, 1931. From Chem. Abst., 26: 1686, March 20, 1932. Precipitation in water mains was found due both to bacteria and to catalytic action of manganese dioxide formed. Essential features for removal are aëration and slight alkalinity. Pyrolusite is effective as catalytic agent and bed of fine coke as aërator and promotor of alkalinity. Cellular structure retains part of manganese dioxide precipitated from water, which in turn acts as catalyzer. Sand filter completes the system, removing precipitated iron as well as manganese. Geological significance of manner of deposition of manganese is discussed.—*R. E. Thompson.*

Stratification of Iron and Manganese in Lake Water in Japan. SHINKICHI YOSHIMURA. Japan J. Geol., 9: 1 and 2, 61–9, 1931; cf. C. A., 25: 4747. From Chem. Abst., 26: 1686, March 20, 1932. Observations on 20 Japanese lakes during period of stagnation confirm previous observation that iron and manganese contents are high in levels containing little or no dissolved oxygen.—*R. E. Thompson.*

Corrosion-Preventing Calcium Carbonate Protective Deposits in Water Pipes. L. W. HAASE. Z. angew. Chem., 44: 990–2, 1931; cf. SCHIKORR, C. A., 25: 1609. From Chem. Abst., 26: 1687, March 20, 1932. Polemical. Factors favoring formation of hard, protective layer of crystalline calcium carbonate, in place of useless amorphous variety, include proper adjustment of $CaO\text{-}CO_2$ equilibrium conditions, velocity of water in pipe, presence of metallic ions, temperature, electrical state, etc.—*R. E. Thompson.*

The Iodine Content of Some Waters of Eastern South Dakota. C. B. STONE. Proc. S. Dakota Acad. Sci., 10: Ser. 27, 35–45, 1927. From Chem. Abst., 26: 1686, March 20, 1932. With two exceptions, water supplies of eastern S. Dakota cities do not contain sufficient iodine to permit quantitative estimation. These two yielded fairly large amounts of iodine, due either to underground springs, or to drainage from lake shores. Results would place S. Dakota in group of states having water of low iodine content.—*R. E. Thompson.*

Pipe Protection. Anon. Western Gas, 7: August, 123–7, 1931; cf. WRESTLER, Western Gas, 7: June, 1931; Pacific Coast Gas Assoc. Proceedings, 512–8, 1930. Chem. Abst., 26: 1753, March 20, 1932. Committee report of Pacific Coast Gas Association. General discussion of ways and means for determin-

ing protection required and of materials required. Some simple relation is sought between soil composition and corrosiveness. Chemical analysis is completely unsatisfactory, but electrical methods are more effective. More common tests are described. Possible reasons for disappointing results are discussed. Several specific materials are considered.—*R. E. Thompson.*

Impermeable Concrete Value in Hydraulic Structural Work. CLIFFORD A. BETTS. Mining Rev., **33**: 3, 15, 1931. From Chem. Abst., **26**: 1744, March 20, 1932. Various factors determining degree of permeability are discussed.—*R. E. Thompson.*

Isolation of Bacillus Coli in the Feces by Means of Urotropine. E. CALISTI. Boll. soc. intern. microbiol. Sez. ital., **3**: 623–4, 1931 (in French). From Chem. Abst., **26**: 1955, April 20, 1932. Method for isolation of *B. coli* in water (cf. Diagnostica tec. lab. (Napoli) No. 6, 1931) was applied to human feces. By adding 1.4 cc. of 10 percent urotropine solution to 10 cc. culture medium, distinct development of *B. coli* colonies was observed, while number of other saprophyte species decreased 80 to 90 percent. Method does not give good results with animal feces.—*R. E. Thompson.*

Solid Sodium Tetrathionate Media in the Bacteriology of Typhoid-Paratyphoid Diagnosis. L. SCHUSTOWA. Zentr. Bakt. Parasitenk., I Abt., **122**: 403–5, 1931. From Chem. Abst., **26**: 1957, April 10, 1932. Agar with pH 7.2 to 7.4 is melted and cooled to 40° to 45°, and to each 100 cc. are added 10 cc. of 50 percent sodium thiosulphate and 2 cc. iodine solution (iodine, 25.0 grams; potassium iodide, 20 grams; distilled water, up to 100 cc.). It is more satisfactory than ENDO's medium.—*R. E. Thompson.*

A New Method for the Detection of B. Coli in Water. D. LUMBAU. Studi Sassaresi (2), **8**: No. 5; Pathologica, **23**: 771–2, 1931. From Chem. Abst., **26**: 1997, April 10, 1932. By adding to culture medium (containing agar, lactose, and 0.005 per cent phenol) 1 cc. of phenol red, 1 cc. of bromocresol purple, and 1 cc. of bromothymol blue, bacilli are always stained yellow on blue ground.—*R. E. Thompson.*

A Self-Releasing Water Sampler for Use Near the Bottom of Lakes. WALDEMAR OHLE. Arch. Hydrobiol., **23**: 690–3, 1931. From Chem., Abst., **26**: 1997, April 10, 1932.—*R. E. Thompson.*

Water Purification. A Century of Progress. HARRISON P. EDDY. Civil Eng., **2**: 82–4, 1932. From Chem. Abst., **26**: 1996, April 10, 1932.—*R. E. Thompson.*

The Use of Active Charcoal in Water Purification. P. SMIT. Génie civil, **99**: 570–4, 1931. From Chem. Abst., **26**: 1996, April 10, 1932. Layer of fine charcoal between 2 layers of sand is better than even granular charcoal.—*R. E. Thompson.*

The Investigation and Evaluation of Active Carbon in Regard to Its Use for Water Purification. M. JAENICKE. Vom Wasser, 5: 83–92, 1931. From Chem. Abst., 26: 1996, April 10, 1932. Active carbon, when used as filter material in water purification, functions in 2 ways: removing (1) organic taste-, odor-, and color-producing substances and (2) residual chlorine from chlorinated water.—*R. E. Thompson.*

Water Purification. Lurgi Ges. für Warmetechnik M. b. H. and FRIEDRICH SIERP. Fr. 715,509, April 16, 1931. From Chem. Abst., 26: 2000, April 10, 1932. Water is purified from contaminating substances which alter taste and odor, e.g., chlorinated phenol, by means of adsorptive material, such as active carbon, containing lower organisms that produce biological action. Active carbon may be treated with sewage water to which phenol or chlorinated phenol is added, and production of active cultures in carbon is accelerated by first treating carbon with solutions of nutrient materials.—*R. E. Thompson.*

Scaling and Corrosion of Steam Turbine. C. N. RIDLEY. Colliery Eng., 8: 329–30, 1931. From Chem. Abst., 26: 1998, April 10, 1932. Priming is caused by dirty feed water, overloading of boilers, oil, or high salt concentration in boiler water. Base exchange treatment, after lime-soda, removes residual calcium and magnesium; but only slight excess of soda should be used. Steam condensate contains dissolved gases and corrosive organic acids. Author advises 80 percent condensate and 20 percent treated water. With little condensate, sodium aluminate should be used and enough soda, or lime, in make-up water to neutralize condensate acids.—*R. E. Thompson.*

Oxidation as a Method for Investigating Sewage Water. S. A. VOZNESENSKIĬ and L. P. ARTEMOVA. J. Applied Chem. (U. S. S. R.), 4: 881–5, 1931. From Chem. Abst., 26: 1999, April 10, 1932. The water is oxidized with chromic acid, both in presence and in absence of ferrous sulfate, and amount of chromic acid consumed is determined at regular intervals. If amount of chromic acid used in oxidizing ferrous sulfate plus sewage water is A; amount of ferrous sulfate, B; and amount used by sewage water, C; then ratio $(A - B)/C$ is constant, K, characteristic for each sewage water and indicative of its purity and of its ease of purification.—*R. E. Thompson.*

Oil Removal from Exhaust and Condensate of Steam Engines. OTTO SCHÖNE. Braunkohle, 31: 61–7, 82–93, 1932. From Chem. Abst., 26: 2036, April 10, 1932. Harmful effects of use of water containing steam cylinder oil for steam raising are discussed. Various types of apparatus for oil removal are described. For treatment of condensate, electrolytic apparatus, chemical methods employing caustic soda and alum to carry down oil particles with aluminum hydroxide, and adsorptive methods using activated carbon are also described.—*R. E. Thompson.*

Method for Determining the Dissolved Oxygen Content of the Mud at the Bottom of a Pond. ARCH E. COLE. Ecology, 13: 51–3, 1932. From Chem. Abst., 26: 2089, April 20, 1932. Apparatus is described by which water can be

collected from mud at bottom of pond without excessive contamination by water above. Experimental results with apparatus indicate relatively minute quantities of dissolved oxygen in mud.—*R. E. Thompson.*

The Determination of the Law of Resistance to the Flow of Water in Pipes. E. PARRY. Proc. World Eng. Cong., Tokyo, 1929, **4**: 151–8, 1931. From Chem. Abst., **26**: 2089. April 20, 1932.—*R. E. Thompson.*

The Electrolytic Corrosion of Underground Metallic Structures by Stray Currents. CLAUDE M. LONGFIELD. J. Inst. Engrs. Australia, **3**: 157–68, 1931. From Chem. Abst., **26**: 2125, April 20, 1932. After describing mechanism of electrolytic dissociation, results are described of some tests made on different soils to determine connection between current density of discharge and amount of corrosion. Nature of results obtained in field surveys and their value for estimation of damage are discussed. Importance of protective coatings for steel mains is stressed. List of references is appended.—*R. E. Thompson.*

Technic for Oxalate Microdetermination of Calcium. L. VELLUZ and R. DESCHASEAUX. Bull. soc. chim. biol., **13**: 797–808, 1931. From Chem., Abst., **26**: 2137, April 20, 1932. Previous technic (C. A., **25**: 3021) is modified to include centrifuging precipitate, washing 3 times with 4 cc. alcohol-ether-water (6:6:5), and once each with alcohol-ether, and ether. This permits removal of ammonium salts, but not of calcium oxalate. Oxalate is titrated cold with potassium permanganate and excess permanganate determined by iodine titration.—*R. E. Thompson.*

Determination of Potassium by Means of Perrhenic Acid. HANS TOLLERT. Z. anorg. allgem. Chem., **204**: 140–2, 1932. From Chem. Abst., **26**: 2138, April 20, 1932. Procedure for determining potassium as K_2ReO_4 is very similar to determination as perchlorate, but appears to be more accurate and more suitable for determining very small quantities of potassium. Advantages, however, are not great enough to warrant use of $HReCl_4$ except in rare cases, because of expense.—*R. E. Thompson.*

Volumetric Determination of Small Quantities of Carbonic Acid. JOSEF LINDER and NORBERT FIGALA. Mikrochemie (2), **4**: 440–5, 1932. From Chem. Abst., **26**: 2141, April 20, 1932. New apparatus is shown for microdetermination of carbon dioxide, which is particularly effective with respect to preventing error from carbon dioxide of air. Results of numerous analyses of 0.3- to 2-milligram portions of sodium carbonate testify to accuracy of modified technic.—*R. E. Thompson.*

Centrifugal Casting of Metals and Alloys. J. E. HURST. Metals & Alloys, **2**: 197–205, 1931; cf. C. A., **26**: 1552. From Chem. Abst., **26**: 2149, April 20, 1932. Difference in properties between centrifugally cast metal and that poured into stationary sand, or chill, molds is explained for cast iron, etc. Distribution of chemical constituents across radial thickness of centrifugally

cast iron pipes 8 and 12 feet long is shown in 14 diagrams. Photographs are included showing centrifugal casting machines and lay-outs of plants for producing castings of this type.—*R. E. Thompson.*

The Strength at High Temperatures of a Cast and a Forged Steel as Used for Turbine Construction. H. J. Tapsell and A. E. Johnson. Dept. Sci. Ind. Research, Eng. Research, Special Rept. No. 17, 1–33, 1931. From Chem. Abst., 26: 2151, April 20, 1932.—*R. E. Thompson.*

Fusion Welding of Pipe Lines. I. I. Trofimov. Groznenskii Neftyanik, 1: 4 and 5, 84–91, 1931. From Chem. Abst., 26: 2160, April 20, 1932. Various defects in acetylene and electric arc welding of pipe lines, caused by overheating, etc., with particular reference to structure and strength of metal are described in detail.—*R. E. Thompson.*

The Sensitivity of E. Coli to Hypochlorite as Influenced by Time of Standing in Water. L. Boyer and J. Boiron. Compt. rend. soc. biol., 108: 982–4, 1931. From Chem. Abst., 26: 2210, April 20, 1932. Viable coli rapidly decrease when sown into distilled water, urea, glucose, peptone, or sewage solution. In peptone and sewage, later growth phase occurs. Sensitivity of organisms to chlorine increases as number of viable organisms decreases.—*R. E. Thompson.*

The Influence of the Oligodynamic Action. H. Gottschalk. Zentr. Bakt. Parasitenk., I Abt., 123: 468–77, 1932; cf. C. A., 25: 1957. From Chem. Abst., 26: 2212, April 20, 1932. Silver salts deposited in filters render water leaving filter bactericidal. Bacterial concentration makes little difference, nor does salt content of water. Increase in temperature increases effect of silver-treated water. Oxygen decreases solubility of silver, but increases lethal action on bacteria.—*R. E. Thompson.*

Graphical Representation of the Composition of Water. R. Frey. Ann. chim. anal. chim. appl., 14: 49–57, 1932. From Chem. Abst., 26: 2258, April 20, 1932. In analysis of mineral water one desires to know content of alkaline earth and alkali carbonates, sulfates, and chlorides rather than content of individual cations and anions. Problem is somewhat simplified by reason of precipitation reactions. Thus there will not be an appreciable quantity of calcium sulfate in solution when excess of alkali carbonate is present. It is shown how different types of mineral waters can be represented graphically by plotting 3 points on triangular diagram.—*R. E. Thompson.*

Amebiasis Outbreak Due to Defective Plumbing in Hotels. Municipal Sanitation, 5: 3, 98, March 1934. Undesirable conditions were revealed in course of survey by committee of two Chicago hotels chiefly concerned, both supplied from city water mains, but requiring supplementary pumpage. Hotel C basement pumps deliver to Hotel C roof tanks and also to upper stories of Hotel A. Hotels C and A have, therefore, a supply in common. Lower stories of both hotels probably obtain their supply chiefly direct from city

mains. Three important types of sanitary hazard existed in both hotels: (1) old, defective piping layouts, lacking provision to render impossible back siphonage from bath tubs and flush toilets into water lines; (2) chance breaks in sanitary sewers, or heavy overflows of mixed sanitary sewage and storm water drainage in and outside of basements; (3) dangerous cross-connections between water and sewer lines, or between lines carrying respectively potable water and water liable to contamination. Danger from (1), old plumbing systems, can only arise in event of continuous overloading by capacity use and is of limited, rather than of explosive, general nature. Concerning (2), following events may have considerable significance. On June 29, unusually heavy rainfall (2.81 inches in 24 hours) occurred, flooding combined sanitary and storm water sewers of city. Flood waters are reported to have penetrated basement of Hotel A through alley manhole and to have reached ice storage house, remaining for several hours. On July 2, second unusual rainfall of 1.63 inches in 24 hours is reported to have caused burst in sewer, from which sewage flowed back into basement of Hotel C and rose to ceiling in icestorage house, submerging much of basement floor and reaching depth of 5 to 6 inches in food handling department. As regards (3), all cross-connections found have since been eliminated. *Epidemiology.* Results of questionnaries sent out by Chicago Board of Health establish that these two hotels were source of most of cases so far traced. Study (stool examination) showed that 3.9 per cent of employees in many hotels and restaurants in Chicago are carriers of *E. histolytica*, or suffer from amebic dysentery, an incidence about equal to that found in general population of United States. At first survey in Hotel C (August 17–September 1), 7.1 per cent of food handlers were found to be infected and were removed from employment as being a possible source of infection for guests and patrons. Cases, however, continued to develop among employees and later surveys in Hotels C and A showed at times more than 18 per cent of employees infected, percentage among non-food handlers being about the same as among food handlers. These facts led to investigation of sanitary conditions. *General recommendations.* (1) Cases of amebiasis (including amebic dysentery) should be promptly reported to health authority. (2) Carriers not engaged in food handling should be treated, but not required to vacate occupations. (3) Feces of untreated and of insufficiently treated carriers and of convalescents, and sometimes those of active cases, contain infective cyst form of parasite and should be disposed of according to local sanitary regulations. Those infected should be informed of manner of transmission of the disease, treated, and cautioned to: (*a*) wash hands thoroughly after using toilet; (*b*) avoid depositing feces where exposed to flies and other insects, or where food, water, or articles handled by other persons, may be contaminated; and (*c*) avoid preparation and handling of food for others until considered parasite-free. Laboratory examination of food handlers should be required if circumstances point to a particular individual or group as a possible source of infection. General examination of all food handlers for *E. histolytica* is considered impracticable. (4) Patient, or carrier, should not be allowed to become a food handler until at least seven days have elapsed since completion of his treatment, followed by three negative examinations of feces at intervals of at least one day. To continue as a food handler thereafter, a

person should have at least four negative stool examinations at intervals of one month. *Recommendations concerning sanitary engineering.* (1) Water and sewer installations should be so arranged as absolutely to prevent contamination of water supplies for domestic use by water of nonpotable quality. Attention is again called to this extraordinary hazard and to necessity of rigid enforcement of these regulations. (2) No physical connection should be permitted between safe and unsafe water supplies. (3) Exhaustive study of water and sewage systems of hotels with antiquated plumbing should be made by competent sanitary engineers, and any defects immediately corrected. (4) Present conditions under which street flood waters can reach and cause sewage to reach basements of hotels should be thoroughly investigated by officials having jurisdiction, and preventive methods adopted.—*R. E. Noble.*

Eijkman's Test on Water Supplies in the Madras Presidency. W. J. Webster and T. N. S. Raghavachari. Indian J. Med. Research, **21**: 525–34, 1934. In series of comparative tests with MacConkey's bile salt lactose broth on waters from a variety of sources, Eijkman test (fermentation of glucose at 46°) was found to be unreliable, samples showing true coli in 0.1 cc. giving, in some instances, a negative Eijkman test in amounts up to 50 cc. Eijkman tubes which showed turbidity and occasionally acidity, but no gas, frequently yielded lactose fermenters on subculture. *B. aërogenes* is inhibited in Eijkman medium to a greater extent than *B. coli.* Streptococci were noted far more frequently in the Eijkman than in the MacConkey tubes.—*R. E. Thompson.*

Calking Stressed Welded Joints Caused Oil-Tank Failure. H. A. Sweet. Eng. News-Record, **111**: 720, December 14, 1933. Collapse of 80,000-barrel, arc-welded oil storage tank of New England Terminal Company, Tiverton, R. I., on November 1, 1933, is attributed by inspection department of Associated Factory Mutual Fire Insurance Companies to calking of highly-stressed, slightly leaking joint, while under pressure. Tank was filled to within 8 inches of top with salt water at time of failure. Recommendations are included.—*R. E. Thompson.*

Safe Welding Practice. H. L. Whittemore. Eng. News-Rec., **112**: 237, February 15, 1934. Brief discussion of report on Tiverton storage tank failure (cf. previous abstract). Welded tanks should not be calked. Defective joints should in every instance be chipped out and re-welded. Most practical method of securing satisfactory welds is to qualify the welders, both before work is commenced and at periodic intervals, say every 6 months. X-ray inspection of welds is not considered reliable.—*R. E. Thompson.*

Boiler Water Conditioning and the Plant Engineer. A. Romayne Moberg. Steam-Plant Engr., **1**: 31–3, 1932. From Chem. Abst., **26**: 2259, April 20, 1932. —*R. E. Thompson.*

The Color and Characteristics of Potable Water. Eugène Lemaire. Gènie civil, **100**: 61–5, 1932. From Chem. Abst., **26**: 2258, April 20, 1932. Slight

changes in temperature of water may greatly change color, because of increased growth of algae. More than 3 p.p.m. of iron imparts taste to water but is not harmful. Dissolved oxygen should not be less than 3 p.p.m. pH of pure water should be 6.7 or 6.8. Drainage from infected areas should be diverted and carefully excluded from collecting reservoir.—*R. E. Thompson.*

The Action of Metallic Copper upon the Bacteria of Drinking Waters. PILOD and CODEVELLE. Compt. rend., **194**: 497–500, 1932. From Chem. Abst., **26**: 2258, April 20, 1932. Water exposed to metallic copper acquires germicidal activity in 4 days at ordinary temperature. Activation is hastened by heat and retarded by cold. Activity is stable to heat, cold, and long standing; it is destroyed by clay, charcoal, peptone solution, and urine. Activated water will kill in 1 or 2 hours colon-typhoid bacilli, streptococci, and staphylococci. It cannot be used for sterilization of drinking water except where prolonged contact with clear water can be assured.—*R. E. Thompson.*

Chlorine Treatment of River Water at Powerton Station. K. E. STOLL. Power, **75**: 168–70, 1932. From Chem. Abst., **26**: 2258, April 20, 1932. Chlorination of condenser cooling water has resulted in average gain in vacuum of 0.24 inch of mercury over period of one year. Gain of 0.06 inch pays for cost of operation and maintenance. No corrosive action has resulted and at no time has it been necessary to take condensers out of service for cleaning.—*R. E. Thompson.*

The Estimation of Steam Boiler Damage and Operation Period by Means of Optical Testing of the Deposits. I. A. SULFRIAN. Vom Wasser, **5**: 162–71, 1931. From Chem. Abst., **26**: 2260, April 20, 1932. Microscopic investigation of boiler deposits is approached in 3 ways: (1) study of crystals present with aid of polarized light; (2) use of certain dyes to identify substances which form lakes with them; (3) micro-chemical analysis. Method of preparing specimens about 0.01 millimeter thick for study is given. While chemical analysis yields only total amount of hydration, micro-analysis enables observer to determine exactly which hydrate, or mixture of hydrates, is present, thus throwing light on local conditions in boiler when deposit was formed. Crystals of anhydrous calcium sulfate in one case showed pseudomorphosis into crystals of calcite, indicating that boiler conditions, as result of variation in load, had oscillated about a condition of equilibrium between solid calcite and calcium sulfate anhydride. Results indicate that character of deposit is determined, not by temperature corresponding to boiler pressure, but rather by local elevated temperature of walls of water tubes, or of boundary surface between deposit and liquid.—*R. E. Thompson.*

Caustic Embrittlement of Steel. I. II. III. F. HUNDESHAGEN. Chem.-Ztg., **56**: 4–5, 17–8, 39–40, 1932. From Chem. Abst., **26**: 2260, April 20, 1932. Literature on caustic embrittlement is reviewed and 33 references are given. Occurrence, causes (American and German theories), and preventive measures are discussed.—*R. E. Thompson.*

The Addition of Hydrochloric Acid to Water for the Prevention of Scale in Water-Jackets and Condensers. H. RUSDEN and J. HENDERSON. J. Chem. Met. Mining Soc. S. Africa, 32: 78–83, 1931. From Chem. Abst., 26: 2260, April 20, 1932. Treatment of cooling water with hydrochloric acid for scale prevention is successful.—*R. E. Thompson.*

Rusting and Dissolving of Iron from Uncoated Iron Pipe of Steam Boilers as the Result of Incorrect Feed-Water Purification. KARL BRAUNGARD. Vom Wasser, 5: 156–61, 1931. From Chem. Abst. 26: 2260, April 20, 1932. Exclusion of free carbon dioxide will not prevent corrosion as long as oxygen is present. Over-softening prevents formation of protective coatings on pipes and thus leaves iron exposed to corrosive action of dissolved oxygen.—*R. E. Thompson.*

Purifying Water. GEORG ORNSTEIN. Ger. 542,343, November 20, 1927. From Chem. Abst., 26: 2262, April 20, 1932. Water is purified by joint action of chlorine and a metal chloride having bactericidal properties, e.g., $CuCl_2$. Apparatus described.—*R. E. Thompson.*

Cement-water Ratio by Weight. INGE LYSE. Eng. News-Rec., 108: 447, 1932. Reply to criticism of MAVIS (C. A., 26: 2568). L. can see no advantage in use of absolute-volume ratios instead of weight ratios as favored by KITTS (C. A., 26: 2568).—*R. E. Thompson (Courtesy Chem. Abst.).*

Intake Vortex Eliminated by Novel Screen Design. T. W. ESPY. Eng. News-Rec., 108: 431, March 24, 1932. Problem of vortex formation at inlet screen of reservoir outlet, with resulting hydraulic losses, was overcome by novel type of screen installed last fall at Crystal Springs reservoir in San Francisco. Principal features of improved design are the placing of screens in horizontal instead of vertical position, increasing area of screened openings to 5 times that of mouth of inlet, and provision for placing guide vanes over top of screens. Old outlet consisted of shaft and tunnels with 3 screened inlets at different elevations. Advantage was taken of extremely low stage of reservoir in 1931 to construct second outlet of same general type, although additional outlet capacity will not be required for several years. Screens previously used were perforated cylinders, 6 feet in diameter by 12 feet high, placed in vertical position over vertical inlets to tunnels. When water was drawn under head of less than 8 feet, vortex would form, becoming more pronounced as water elevation was lowered, amount of air entrapped seriously decreasing capacity of pipes and pumps. New cylindrical screens are 60 feet in diameter and 44 feet long, and are provided with T-connection at center of lower side, in form of metal sleeve which projects 2 feet into mouth of inlet. Angles welded along sides provide support for vanes, if these shall, in future, be considered advisable. Vanes extend 3 feet above top of screen, and, in effect, cause water to enter screen through 6 compartments. Screens may be lifted off and replaced by covers, if it is desired to unwater tunnels or shaft. New outlet has not been used, but screen of new design, installed to replace lowest of 3 screens on old inlet, which was missing, presumably shaken off in earth-

quake of 1906, was found to function perfectly. There was no tendency for vortex to form, even when water was drawn down to top of screen.—*R. E. Thompson.*

Underground Water Rights. SHELDON K. BAKER. Eng. News-Rec., **108:** 483, March 31, 1932. Discussion of decision of Supreme Court of Arizona in case of Southwest Cotton Company *vs.* Maricopa County Municipal Water Conservation District No. 1. Writer believes that, instead of hastening development in state by removing menace to utilization of such surface water supply as is still undeveloped, it places serious obstruction in way of development of resource far more important economically, viz., the great underflow of valleys of southeastern Arizona. Law is so inadequate for present-day conditions, that in order for development to continue, there must be revision of water code that will give to underground waters the standing in law to which they are entitled by all rules of common sense.—*R. E. Thompson.*

Taste Problems in Ontario Water Supplies. E. W. JOHNSTON. Cont. Rec. and Eng. Rev., **46:** 264–7, 1932. There are 237 municipal water supplies in Ontario, serving 281 communities. Objectionable tastes have caused difficulties in 58, or 24 percent, of these. List is given of 29 municipalities in province where taste troubles have been prevalent, showing source of supply, nature of tastes complained of, and corrective treatments tried. Pre-ammoniation, using $(NH_4)_2SO_4$, and addition of powdered activated carbon have been found most effective remedies.—*R. E. Thompson (Courtesy Chem. Abst.).*

Tower-Excavator Development for Levee Building. J. C. FRENCH. Eng. News-Rec., **108:** 474–599, March 31, 1932. Development, operation, etc., of traveling-tower slackline-cableway excavator, developed specifically for levee construction on Mississippi River, described and discussed.—*R. E. Thompson.*

Streamflow from Rainfall by Unit-Graph Method. L. K. SHERMAN. Eng. News-Rec., **108:** 501–5, April 7, 1932. From single observed hydrograph, referring to storm lasting one day, it is possible to compute for same watershed runoff history corresponding to rainfall of any duration, or degree of intensity. Method of determining the unit graph for any drainage area and its application to computation of hydrograph for any individual storm, or sequence of storms, of any duration, or intensity, over any period of time, is described and discussed.—*R. E. Thompson.*

Welding Methods for Header Fabrication. Cont. Rec. and Eng. Rev., **46:** 332–6, March 23, 1932. Detailed, illustrated discussion of fabrication of piping system headers by oxyacetylene welding.—*R. E. Thompson.*

Water Purification. GEORGE G. NASMITH. Can. Pub. Health J., **23:** 59–65, 1932. General discussion of water purification. Interesting observation made at Windsor, Ont., is outlined. During summer months algae seriously interfere with filtration, filter runs being reduced to 3 hours, or even less.

Coagulation with lime and iron increased filter runs to 7 or 8 hours. Addition of 0.1 p.p.m. chlorine prior to coagulation with alum caused floc to form in strings up to 1 inch in length, which settled rapidly, and prolonged filter runs to 60 or 70 hours.—*R. E. Thompson (Courtesy Chem. Abst.).*

Model Tests Confirm Design of Hoover Dam. J. L. SAVAGE and IVAN E. HOUK. Eng. News-Rec., **108**: 494–9, April 7, 1932. Extensive tests of model of Hoover Dam have been carried out by Bureau of Reclamation at University of Colorado in co-operation with Engineering Foundation arch dam committee. Tests made and results obtained are given and their interpretation discussed. Results checked remarkably well with design calculations, and, while not yet complete, have already set at rest the principal questions affecting the design. —*R. E. Thompson.*

Installing Water Mains in Perpetually Frozen Ground. J. CLARK KEITH. Cont. Rec. and Eng. Rev., **46**: 350–1, March 30, 1932. Extreme conditions in Churchill, Manitoba, terminus of Hudson Bay Railway, are outlined. Holes drilled 100 feet to rock have shown soil to be frozen to that depth. Under the moss, ground thaws only to depth of one or two feet. Mean annual temperature is below 20°F. To secure water supply, Department of Railways and Canals has decided to construct reservoir deep enough to store sufficient water under the 7 or 8 feet of ice that would form on surface in winter. Pipe line has been constructed above ground level on small piles and is being covered with moss to depth of 4 or 5 feet. In all probability, town itself, when it develops, will draw water from pipe line in heated boxes above ground. If pipe line fails to function in winter months, citizens will have to depend on ice cut from lakes and melted.—*R. E. Thompson.*

River Tunnel in England Made Dry by Grouting. Eng. News-Rec., **108**: 430, March 24, 1932. A 1-mile section of 4½-mile tunnel of Great Western Railway under tidal part of Severn River in England has always given considerable trouble owing to seams and faults. This section was scene of great inrushes of gravel during its construction in 1879 and 1882. In 1924 and again in 1929, excessive leakage occurred, due to development of open joints in marl above tunnel, but in both cases mouth of "pipe" was accessible at low tide and it was possible to seal opening with concrete poured from above. Construction records and recurring infiltration of water proved existence of open joints and faults in surrounding strata, and it was decided to strengthen lining, fill voids and stop flow of water along roof, and consolidate ground by cement grouting. First 0-mile section was completed in April, 1930, and work was so successful that it was decided to extent it for another half-mile. The two ventilating shafts were also grouted. Tunnel work required 4,233 holes, aggregating 21,683 feet and 8,132 tons of grout, or 1.48 tons per foot of tunnel. Work was described recently in Railway Engineer by RAYMOND CARPMAEL.—*R. E. Thompson.*

Directory of Important Waterworks Systems of Canada. Cont. Rec. and Eng. Rev., **46**: 276–84, 1932. Authoritative and up-to-date statistics concern-

ing water supplies of leading cities and towns throughout Canada.—*R. E. Thompson.*

Waterworks and Sewerage Programs for 1932. A. E. Berry. Cont. Rec. and Eng. Rev., **46**: 262–3, 267, March 9, 1932. Works in progress or projected in Ontario are outlined.—*R. E. Thompson.*

Waterworks Stability Proved in Time of Depression. Eng. News-Rec., **108**: 606–8, April 28, 1932. Data given regarding effect of depression on consumption, revenue, delinquents, financial policy, etc., in New York, Chicago, Detroit, St. Louis, San Francisco, New Orleans, Indianapolis, Memphis, and Minneapolis. Revenues have decreased but little: in very few cases to extent necessitating curtailment of activities. In all cases consumer service is being maintained unimpaired, and maintenance of existing facilities is not being neglected. Capital expenditures for distribution system extensions and plant improvement, however, have been deferred in many instances because of critical financial condition of city proper. In most cities reported upon, consumption declined somewhat in 1931, but in Minneapolis both consumption and revenue increased.—*R. E. Thompson.*

Design of Casing Pipe for Pipe Undercrossings. M. G. Spangler. Eng. News-Rec., **108**: 580–1, April 21, 1932. Design of steel casing used to surround gas and oil lines at railway and highway crossings is discussed and specifications developed by Iowa Engineering Experimental Station at request of Iowa Railway Commission are given.—*R. E. Thompson.*

Dependable Hydrants for Adequate Fire Protection. Clarence Goldsmith and George Tatnall. Eng. News-Rec., **108**: 611–3, April 28, 1932. Detailed discussion of basic requirements for fire hydrants, their installation, location, maintenance, etc. Importance of regular inspection is emphasized. Forms for records of inspections should be drawn up with spaces for checking all necessary data that inspector notes. Location should be entered before inspection from office records, and hydrants should be numbered and routed so that none will be missed. Street connections should not be less than 6 inches in diameter. When replacing 4-inch connections, it has been found satisfactory to use 4- to 6-inch increaser instead of replacing 4-inch tee in main. Location, inspection, etc., of valves in distribution system are also discussed. Non-uniform valves, which it is not practicable to replace, should carry some distinguishing mark in addition to office records; probably simplest is red paint on inside of box cover.—*R. E. Thompson.*

Powering the Water Plant with Modern Equipment. Charles Brossman. Eng. News-Rec., **108**: 613-6, April 28, 1932. Discussion of basic factors influencing selection and operation of pumping station equipment. Author reviews progress made in operation of large power stations and suggests application of methods employed in that field to water works pumping stations. One ton of coal yields from 10 to 20 per cent more power than it did 10 years ago and, in general way, this increased efficiency holds good for all available

sources of power. Mechanical stokers are available for boilers from 75 horsepower up and can be considered even for small installations, as usual hand-fired boiler plant does not give good efficiencies. Ordinary stoker and water-tube boiler installation should give (with average good coal) approximately 72 per cent combined stoker and boiler efficiency; air preheaters will increase efficiency to about 85 per cent. Efficiencies of modern stoker and of pulverized fuel equipment will not be found materially different when all factors are considered. Latter may be found adaptable to water works conditions. Consideration is now being given to purchased electrical power in many small and medium-size pumping plants.—*R. E. Thompson.*

Now is the Time to Make Waterworks Improvements. MALCOLM PIRNIE. Eng. News-Rec., **108:** 609–11, April 28, 1932. Plea for construction of public works as step toward economic recovery. Needed public works offer greatest opportunity to increase employment. On one hand deferred works are accumulating all over country at time when they could be undertaken at less than normal cost, and on other hand unproductive, but necessary, expenditures are being made for relief of unemployed. Needed water works betterments stand out at head of list of public works that are financially sound and productive of benefits worth their cost or more. Water works betterments to value of at least $500,000,000 could be proceeded with without borrowing from normal future growth and extensions. Attitude of investment bankers toward municipal securities has greatly retarded such activities. Graphical data on bond issues for public works since 1905 are given indicating that there has been but reasonable provision of facilities and services in contradiction to widely heralded claims that municipalities have been on wild orgy of spending.—*R. E. Thompson.*

Industrial Railway Equipment for Levee Building. J. C. FRENCH. Eng. News-Rec., **108:** 582–4, April 21, 1932. Illustrated description of combination of dragline-excavator and industrial railway equipment developed on Mississippi River levee work.—*R. E. Thompson.*

Snow Records in California Indicate Good Water Year. Eng. News-Rec., **108:** 559, April 14, 1932. Survey made in latter part of February at key snow courses and available precipitation data to March 1, indicate depth and water content of snow to March 1 to be from 2 to 4 times greater than that of year ago. Tabulated data are given comparing present and last season's snow by stream basins.—*R. E. Thompson.*

Slab-Revetment Equipment, Mississippi River. BREHON SOMERVELL. Eng. News-Rec., **108:** 554–7, April 14, 1932. Illustrated description of bank grading equipment and elaborate plant for fabrication of mattresses composed of large precast concrete slabs.—*R. E. Thompson.*

Automatic Material Batchers at Hoover Dam. Eng. News-Rec., **108:** 534–7, April 14, 1932. Illustrated description of automatic aggregate batching equipment installed by Six Companies, Incorporated, at Hoover dam, as part

of elaborate concrete mixing plant built for manufacture of nearly 4,500,000 cubic yards of concrete required for dam, tunnel lining, spillway, and control and diversion works. The 4 mixers have combined capacity of 280 cubic yards per hour. Batching equipment was required to handle 5 different sizes of aggregates (sand; fine, intermediate, and coarse gravel; and cobbles) and also cement and water. Consistency indicator for each mixer is located immediately before operator, and each batch is checked for consistency before discharge. Separate ingredient weights and consistency are recorded automatically for every batch.—*R. E. Thompson.*

A Tale of Construction Marvels at Hoover Dam. Eng. News-Rec., **108**: 570–4, April 21, 1932. Interesting illustrated description of progress on Hoover dam as seen on visit to site one year after activities commenced.—*R. E. Thompson.*

North Creek Water Supply, N. Y. Eng. News-Rec., **108**: 602, April 21, 1932. Unit prices given from contract for construction of earth dam and storage reservoir, 100,000-gallon steel storage tank, cast iron mains and supply main, and chlorinator recently let by North Creek Water District. System will supply North Creek and Holcombville, N. Y. Dam, 280 feet long by 120 feet wide at base and 39 feet high, forms reservoir of 20 million gallons capacity on Roaring Brook.—*R. E. Thompson.*

Economy in Wider Use of Protective Coatings on Pipes. William Thompson Smith. Eng. News-Rec., **108**: 576–8, April 21, 1932. Detailed discussion of economics of protective coatings on oil and gas pipe lines, which involve considerable expenditure which, while small compared with total cost of line, is nevertheless difficult to justify, except at "hot spots." Recently developed methods of determining soil corrosiveness, although not absolutely accurate, permit allocation of limited appropriations for coatings to sections where protection is most necessary. Many designers prefer to lay lines without protection and await development of corrosion and leaks as infallible indication of locations where protection is required. In many cases this policy is wrong: expected saving is non-existent: at times it proves more costly. Effect of major factors bearing on economics of this procedure is subject to close determination based on reasonable assumptions, though this is seldom done. There are many other considerations, however, effect of which cannot be accurately estimated in advance. Only small percentage of total length of bare line can be reconditioned before total cost will equal, or exceed, that entailed by complete initial protection. Data are given indicating that in case of 8-inch line, neglecting interest charges, this percentage is only about 10 per cent. Chart is given for graphical solution of problem of determining length of line which may be reconditioned before cost exceeds that of initial complete protection, taking into consideration 4 factors involved, namely, cost of protection, cost of reconditioning, interest rate, and age of line at time of reconditioning. It is shown that increase in age of line at time of reconditioning from 2 to from 6 to 10 years does not greatly increase percentage which can be economically reconditioned. Best method of reducing protection costs is to

apply coating in all known, or suspected, areas of corrosion and omit such protection only in areas ascertained by thorough and reliable soil-corrosion survey to be non-corrosive.—*R. E. Thompson.*

Microdetermination of Calcium Ion. M. MOUSSERON. Bull. biol. pharm., 1931: 123–7; cf. C. A., 25: 2076. From Chem. Abst., 26: 2394, May 10, 1932. Review of several papers by author and associates.—*R. E. Thompson.*

Tri-State Compact Recommended for Control of Pollution at New York. Eng. News-Rec., 108: 574–5, April 21, 1932. Identical bills for creation of interstate sanitation district and establishment of interstate sanitation commission for control of future pollution and abatement of existing pollution on tidal and coastal waters in and adjoining New York Metropolitan District were presented to New York and New Jersey legislatures for action this year and similar bills will be presented to Connecticut legislature at next session in 1933. Bills have been passed by New York legislature and signed by governor and will become effective upon approval of bill presented in New Jersey, in so far as it affects waters within these states. If Connecticut desires to become party to compact, its approval will make possible extension of treaty area to include that part of Connecticut shore area contributing large amount of pollution to Long Island Sound. Area included under provisions of compact is shown on map. Jurisdiction of proposed sanitation commission will include "all coastal, estuarial, and tidal waters within, or covering portions of, signatory states and those portions of all towns, cities, boroughs, and villages that border upon and the natural drainage from which is tributary to such tidal water, together with all areas in signatory states, the artificial drainage, sewage or sewage effluents of which now discharge or may hereafter discharge through artificial outlets to such tidal waters." Proposed compact calls for appointment of 5 commissioners by each of the two states, or the three states if Connecticut becomes party to it. This sanitation commission is empowered to determine int which of two classes (on basis of whether or not the designated water areas are, or are not, expected to be used primarily for recreational purposes, shellfish culture and development of fish life) all waters under its jurisdiction are to be put, the two general classifications being prescribed in compact. It is also empowered to issue orders requiring such treatment of sewage and industrial waste deemed necessary to obtain or maintain purity required, and to stop contamination of waters under its jurisdiction by injunction proceedings. Commission will have no power to undertake construction of sewage disposal plants or other work to put its orders into effect: responsibility for making its orders effective is assumed by the states. Conditions under which sewage or other polluting matter may be discharged into tidal waters are given. Signatory states also agree that water flowing into compact area waters in tributary streams shall be maintained up to standards laid down by commission for waters into which such streams discharge.—*R. E. Thompson.*

Earthfill Dam Saved from Destruction by Cutting Through Side Embankment. Eng. News-Rec., 108: 561–2, April 14, 1932. Undercutting of spill-

way paving of Hunter's Creek dam, a rolled-fill earth embankment, 40 feet high and 500 feet long, at Northville, N. Y., beginning on March 29 when creek was in flood, progressed so rapidly as to endanger dam itself and highway bridge over spillway and ultimately resulted in cutting of artificial channel through embankment to check flow through spillway channel. Cut saved spillway and bridge but resulted in scouring out of channel 150 feet wide and 45 feet deep. Dam was constructed recently by Hudson River Regulating District to stabilize water level in arm of Sacandaga reservoir at Northville. Surface material at point at which cut was made was coarse gravel, but when cut was opened it developed that fine material lay below the heavy gravel. This material cut through so rapidly that pond, estimated to hold 32 million cubic feet, was emptied in 1 hour. No damage resulted as flow was absorbed in larger reservoir below. One engineer lost his life in attempt to determine how deeply caving had cut back under spillway paving. Dam will be completely restored as soon as practicable.—*R. E. Thompson.*

Pitting of Hydraulic Turbine Runners. ALBERT H. MYERS. Elec. World, **99**: 328–9, 1932. From Chem. Abst., **26**: 2404, May 10, 1932. Examples of installations that exhibit corrosion pits from spot oxidation and from electrolytic action are noted. Causes and corrective measures are pointed out. One cause is electrolytic action accentuated by impure water or stray electric currents.—*R. E. Thompson.*

NEW BOOKS

Wasseraufbereitung (Water Purification). Handbook of the Bamag-Meguin Aktiengesellschaft, Berlin. 58 pp. Subject is competently treated from point of view of plant practice, with special reference to equipment manufactured by above concern. Following items are discussed: coagulation and filtration; rakes and screens; removal of iron, manganese, and carbon dioxide; softening with lime and soda ash; swimming pool water; chlorination; and use of activated carbon. Types of wet- and dry-feed machines, slow and rapid mixing devices, filters and accessories, control tables, screens, aërators and de-gasifiers, swimming pool equipment, chlorine feed and control devices, etc. are sufficiently varied to suit the needs of any plant. Booklet is abundantly illustrated with photographs and diagrams of equipment and installations.—*Selma Gottlieb.*

Selected Papers, The Eleventh Annual Michigan Conference on Water Purification, Grand Rapids, Michigan, September 19–20, 1933. Mimeographed Separates, 8½ by 11 in. J. M. HEPLER, Secretary, State Department of Health, Lansing, Michigan. **Ammonia-Chlorine Treatment Yields Nitrites in Effluent.** ROBERTS HULBERT. 6 pp. Nitrites in effluent of rapid sand filter, due to action of nitrifying bacteria in the bed encouraged by ammonia treatment, have been found to exist at several water filtration plants in Detroit area. Because of their high chlorine demand, nitrites consume much chlorine in oxidation to nitrates. The *ortho*-tolidine test gives misleading results, since nitrites also yield a yellow color. Waste of chlorine can be avoided by using

sufficient ammonia with pre-chlorination process to allow free ammonia to carry through filters and combine with post-chlorine to form a chloramine. A 10-minute contact period for the ortho-tolidine test with samples stored in dark gives true residual chlorine, as nitrite color is slow to develop and is promoted by sunlight. **Chemical Action Between Filter Media and Hot Alkaline Solution.** H. G. TURNER and G. S. SCOTT. 5 pp. Tests of Ottawa filter sand and "Anthrafilt" (20 x 30 mesh anthracite) indicate that when subjected to 1 per cent NaOH solution for 30 hours at 10 atmospheres pressure for 13 times in succession, Ottawa sand lost 64.7 per cent of its original weight, while Anthrafilt lost 6½ per cent only. Behavior of filter media with alkaline solutions is significant in consideration of industrial water treatment plants employing hot process methods. **Notes on the Ferric Chloride Coagulation of Sewage.** E. F. ELDRIDGE. 4 pp. Ferric chloride added to water in normal pH range hydrolizes, forming hydrates of ferric oxide. Flocculation of these hydrates can be promoted by adjustment of pH, or by neutralization of positive charge carried by the colloid, either by addition of colloid of opposite charge, or by adsorption of an ion of strongly opposite charge. This behavior is indicated by laboratory experiments. In actual tests with sewage, mixing or flocculation period of about 40 minutes was found desirable with which ferric chloride coagulation proves a valuable adjunct to settling. **Handling Beet Sugar Wastes in a Municipal Water Treatment Plant.** W. D. LOREAUX. 6 pp. Dundee, Michigan, raw water supply is taken from River Raisin, on watershed of which beet sugar factory discharges wastes from October to June. Carbon filter unit failed to eliminate beet sugar wastes during last months of first year's run, or thereafter. By means of pre-chlorination and application of powdered activated carbon, satisfactory effluent from river water carrying beet washings has been obtained. By means of pre-ammoniation, pre-chlorination, and powdered activated carbon good results have been obtained on river water carrying Steffens wastes. Ratio of chlorine to ammonia 7:1. Results indicate that powdered activated carbon treatment supplementing pre-ammoniation and pre-chlorination is economical, gives flexible type of treatment, sweetens settling basin sludge, and almost completely removes objectionable tastes and odors from river water containing beet sugar factory wastes. **The Conversion of a Plain Sedimentation Basin to a Dorr Clarifier.** JOE DEKORN. 4 pp. Description of changes made in groined arch settling basin, 125 feet wide, 140 feet long, and 13 feet deep, adapting it to accomodate 122-foot diameter Dorr clarifier.—*R. L. McNamee.*

An Account of Some Experiments of the Water and Sewage Purification Committee, Madras, on the Treatment of Stored Tropical Water. H. H. KING, T. N. S. RAGHAVACHARI and K. V. NARASIMHA IYENGAR. Transactions, 8th Congress of the Far Eastern Association of Tropical Medicine, 151–82, December, 1930 (1931). Studies of the committee and of others on purification of Madras water supply are reviewed and discussed rather extensively. Supply, which is obtained chiefly from Kortaliar River, is stored in two reservoirs (at least 6 months' storage capacity is required, owing to the dry season) and purified by slow sand filtration. Filters do not function satisfactorily and hydrogen sulfide is produced during warmer months in sand beds and in

distribution system, due, it is believed, to putrefaction of organic matter. Filaments of Crenothrix and small brown masses of vegetable debris are common in the distributed water. Prechlorination has been of value in lengthening filter runs. Chlorination subsequent to filtration, with reasonable dosages, is ineffective, owing to presence of hydrogen sulfide. Extended experiments have been made on alum coagulation and on use of rapid sand and percolating filters prior to slow sand filtration. The percolating filters have given best results from standpoints of removal of organic matter and lactose-fermenting organisms and of freedom from hydrogen sulfide. Fourteen references.—*R. E. Thompson.*